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The development of a modified soil shear vane

Vernelle Truman Smith

THE DEVELOPMENT OF A MODIFIED SOIL SHEAR VANE

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A

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Approved by

Thomas S. Fry (advisor) William John Murphy

John B. Heagler Jr. Edward J. Beckemeyer

ABSTRACT

The purpose of this investigation was to determine how the soil shear graph could be modified to measure more accurately values of shear strength of fine-grained, cohesive soils. Several modifications were tried, however, the one that gave the most satisfactory results consisted of a vane and base plate mechanism.

A red clay of high plasticity obtained from a local cave was tested in both the natural or undisturbed state and in the remolded state. The only variable was the moisture content. Compaction effort was the same for all remolded specimens. Tests were conducted on undisturbed samples taken with thin walled shelby tubes.

The results obtained from testing with the modified soil shear graph were compared with the shear graph as indicated by the vane shear apparatus, unconfined compression test and the standard soil shear graph.

ACKNOWLEDGMENT

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Acknowledgment is also due Professor John B. Heagler, Jr. for his helpful recommendations and to John D. Smith for his assistance in producing the various mechanical innovations to the shear graph which were tried.

Thanks are also given to Captain Byron Walker, Captain Guy Payne and Marvin Byington for their assistance in obtaining soil samples and collecting experimental data.

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I. INTRODUCTION

In all soil stability problems, which include the design of embankments, foundations or retaining walls, a knowledge of the internal strength of the soil mass involved is necessary in order to provide the engineer with the fundamental background for evaluating and designing his structures. Simply stated, adequate foundation support is a function of the inherent or built in resistance to shearing deformation of the underlying soil mass.

Due to the complex structure and composition of soils, the normal methods of analysis used for determining shear strength of other materials do not apply. No universal method exists at the present time by which accurate, stress-strain relationships for soils can be determined. This is especially true in the case of clays and clayey soils. Clay is quite different from any other fraction of the soil both in internal crystalline structure and shape characteristics. The clay mineral grains are laminated and exhibit a platey structure as compared with bulky shaped minerals of the non-plastic soils.

There are several types of equipment and procedures available for measuring the shear strength of soil, among these are the bevameter, direct shear test, torsional shear test, triaxial test, unconfined test, British laboratory vane shear apparatus, cone penetrometer test and the recently developed soil shear graph. Of all these available types of equipment, only three can be used to test the soil in situ. They are the vane shear apparatus, cone penetrometer and soil shear graph. Of these, only the cone penetrometer and soil shear graph can be carried and operated by one man while conducting a field survey or preliminary soils investigation.

All of these methods are reliable and fairly accurate when used to test friction-cohesive soils, however, none of them produces the accuracy or reliability required when used on fine-grained cohesive soils. Since the development of the shear graph some research has been done to determine the correlation between the shear strength as indicated by the shear graph and the unconfined compression test.(1) The results of this investigation only made it more apparent that a more reliable piece of equipment is desirable.

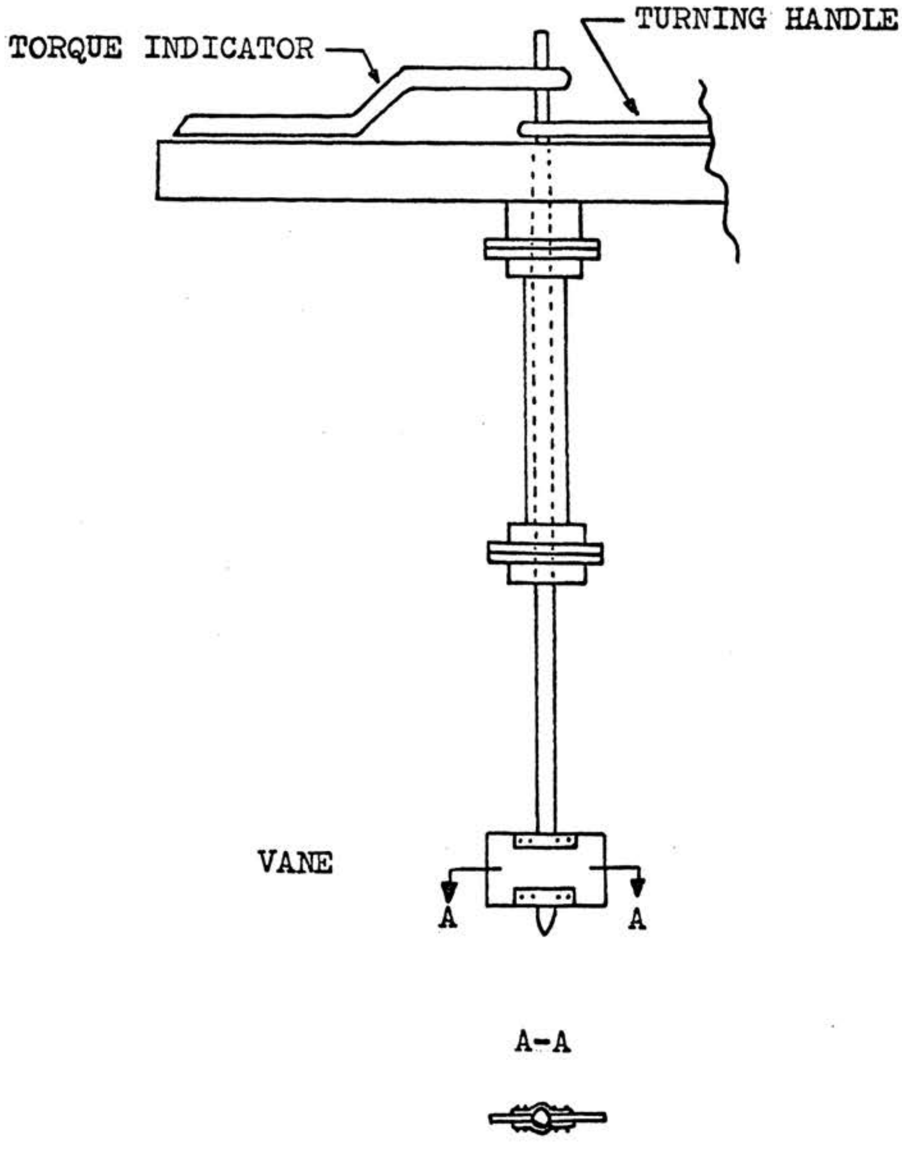
The objective of this investigation is to determine whether the soil shear graph can be modified to give more reliability and accuracy and to correlate the results of shear tests made with a modified shear graph with the results as indicated by the British laboratory vane shear, unconfined compression test and the standard soil shear graph as it presently exists.

II. REVIEW OF LITERATURE

Much interest has been exhibited in recent years in the use and value of the vane borer for measuring the in situ shear strength of soils at varying depths. With the vane borer apparatus the shear resistance is determined by measurement of the torque required to shear the soil stratum with vertical blades attached to a shaft. Torque is applied either by hand or mechanical methods and the resulting shear strength is computed.

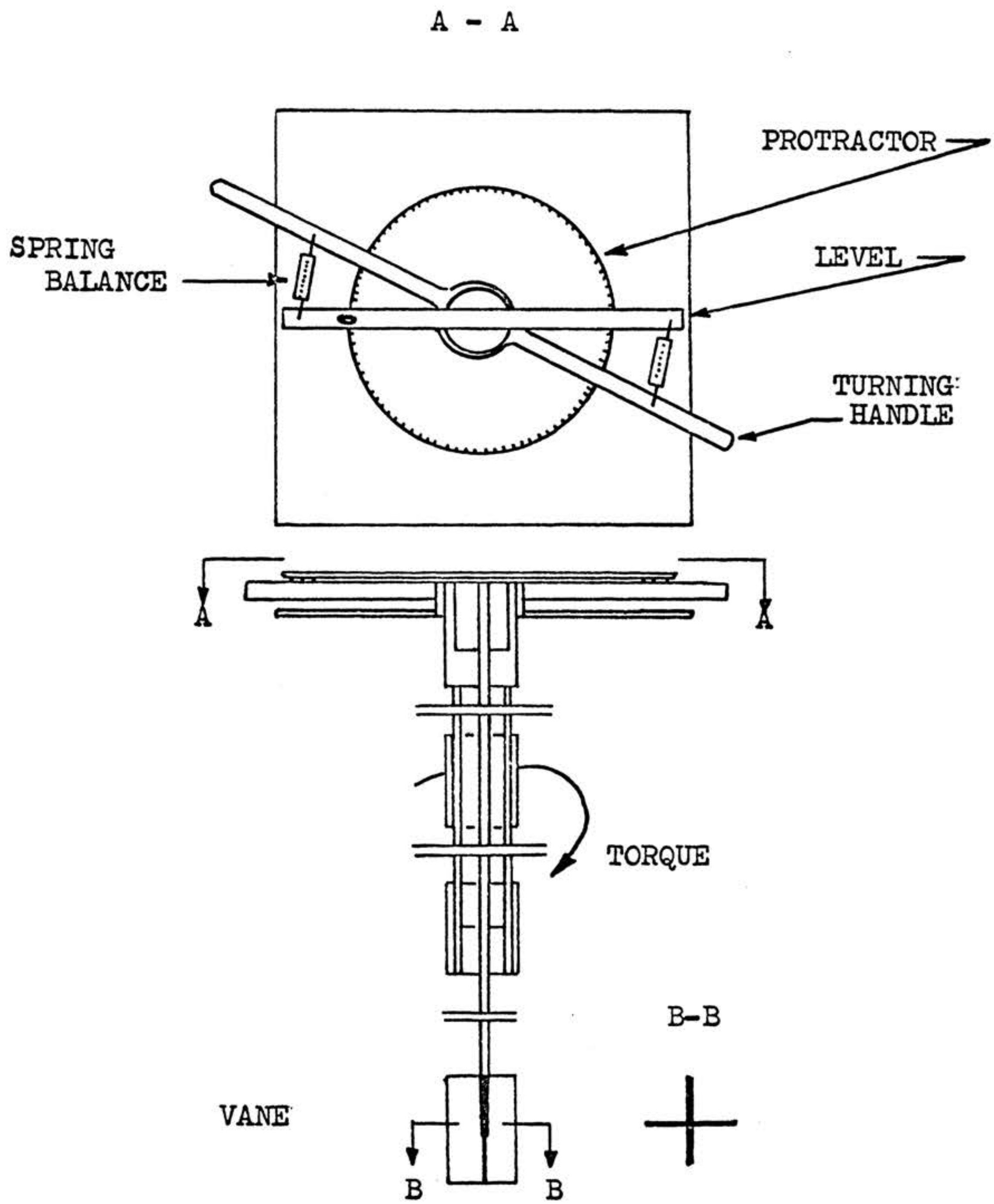
The earliest indications in the literature of a vane borer being used to determine the values of soil shear strength in situ date back to 1928.[2] J. Olsson and Professor Carl Forssell of Sweden and Deutsche Forschungsgesellschaft fur Bodenmechanik (Degebo) of Germany[2][3] were probably the first to conduct experiments with this device. It is reported that in these tests, especially those conducted in Germany, that full consideration was not given to the sensitivity of the clay. In 1930, Professor Forssell demonstrated a vane apparatus at the Third International Congress for Applied Mechanics in Stockholm.[2] A sketch of one of these early vane borers is shown in Figure 1.[4]

It was not until 1947 that the Royal Swedish Geotechnical Institute reopened the files and undertook a program of experimentation to determine methods of improving the vane borer apparatus. The vane borer used in these experiments was designed by Mr. Torsten Kallstenius, the mathematical analysis was performed by Mr. Sten Odenstad and the actual testing was conducted under the supervision of Mr. Lyman Cadling and Mr. Nils Flodin. After experimenting with Vane Borer Type I as shown in Figure 2,[2] a vane borer for practical use was constructed.



EARLY VANE BORER

FIGURE 1.



VANE BORER TYPE I

FIGURE 2.

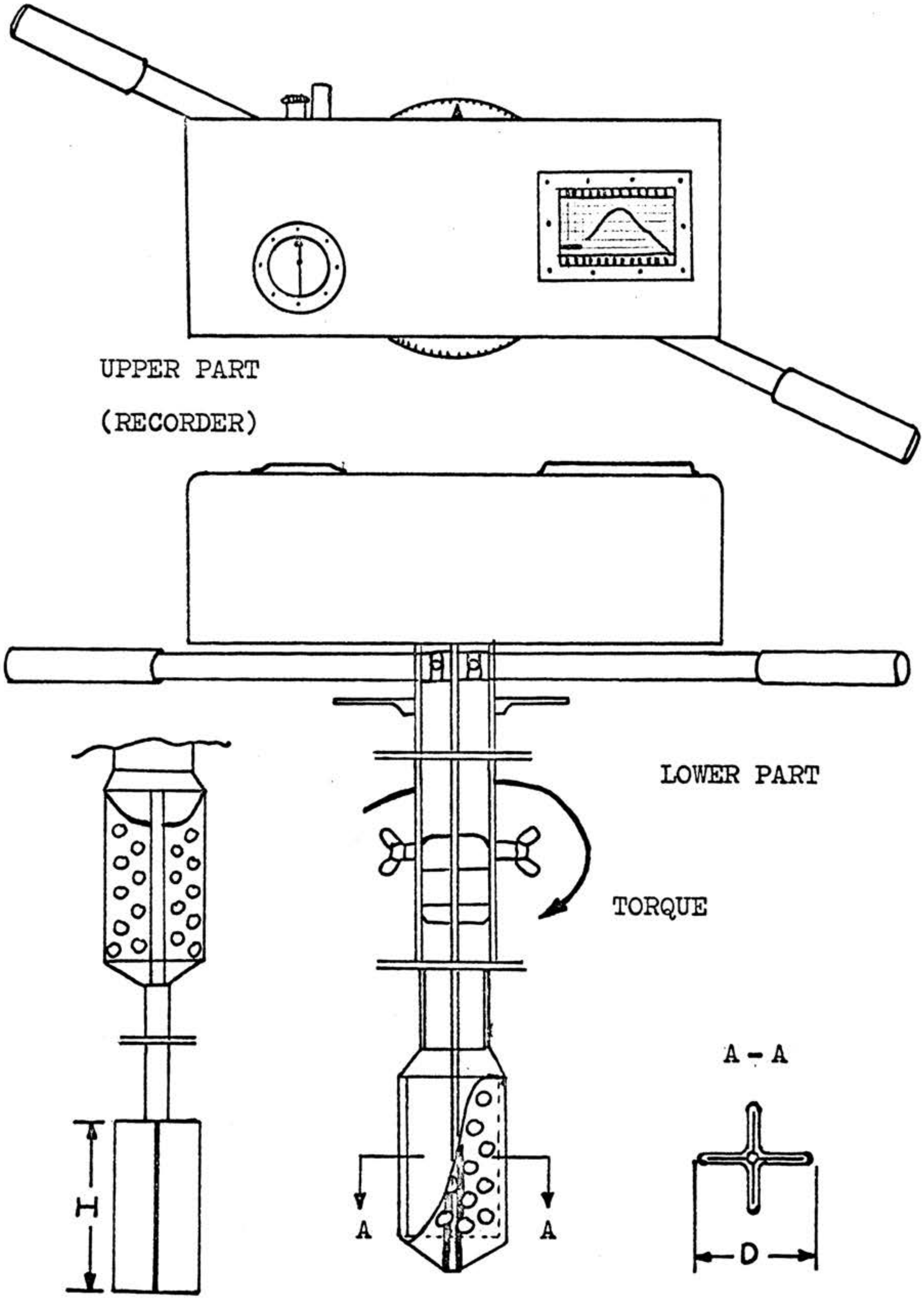
This final design, called Vane Borer Type II, consisted of two main parts. The lower part, which is driven into the soil, consists of an inner and outer system. The outer system was designed to protect the inner system during driving operations. The inner system is comprised of vanes at the bottom of a shaft which in turn is connected to the upper part of the instrument at the ground surface. The upper part of the instrument contains the equipment for measuring and recording the torsional moment applied. Figure 3[2] illustrates the Vane Borer Type II.

By means of a torsion bar, a torque is applied to the shaft which in turn imparts a torque to the vanes. A constant speed spring motor slowly moves a slip of paper so the moment is continuously recorded. A bell, which is connected to the spring motor, and a protractor were added to insure that a constant rate of rotation was applied. In this model the torsion bar is rotated by hand.

Such variables as vane dimensions, length and diameter of shaft, rate of rotation and number of wings of the vane were investigated. The final decision was to use a four winged vane with a height to diameter ratio of 2, $\left(\frac{H}{D} = 2\right)$. A rate of 0.1 deg/sec was selected and the shaft was constructed so that a minimum amount of applied torque was required before stress was transferred to the vanes and the soil.

The soils tested included fine sand, sand with layers of clay, clay with layers of sand and pure clay. The results of using the vane borer compared favorably with the values determined by the falling cone and unconfined compression test.

The shear strength calculations were based on the following assumptions:[2]



VANE BORER TYPE II

FIGURE 3.

1. The surface of failure is a circumscribed cylinder of the same dimensions as the vane used.

2. The stress distribution at maximum torsional moment is uniform over the entire surface area including the ends.

3. There is negligible friction in the borer mechanism.

4. The torsional moment exerted by the clay on the shaft of the borer is negligible.

5. The maximum torsional moment (M_{max}) at the instant of failure is:

$$M_{max} = S\left(\pi D H \frac{D}{2} + \frac{2\pi D^2}{4} \cdot \frac{2}{3}\frac{D}{2}\right) \text{ and if } H = 2D \text{ then}$$

$$S = \frac{6}{7}\frac{M_{max}}{\pi D^3} = \frac{M_{max}}{C}$$

where: S = shear strength

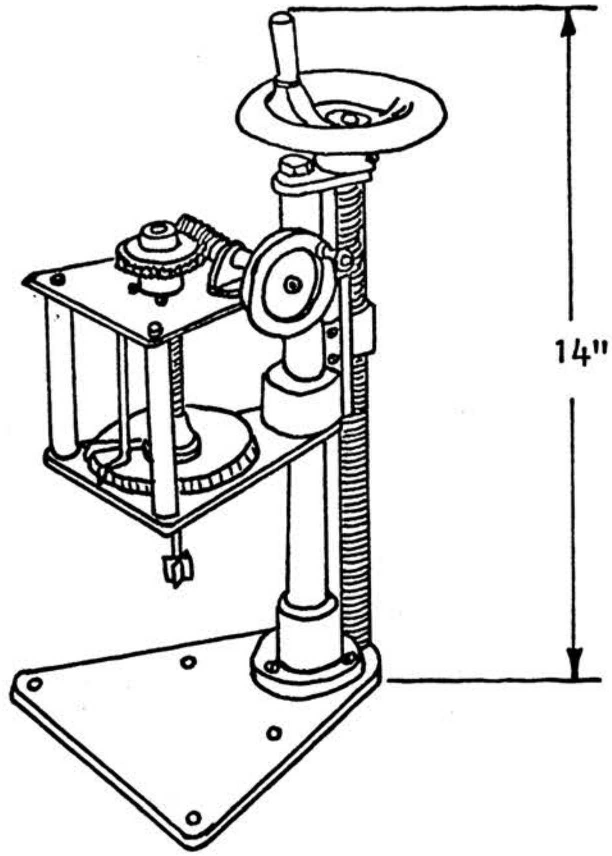
D = diameter of vane

H = height of vane

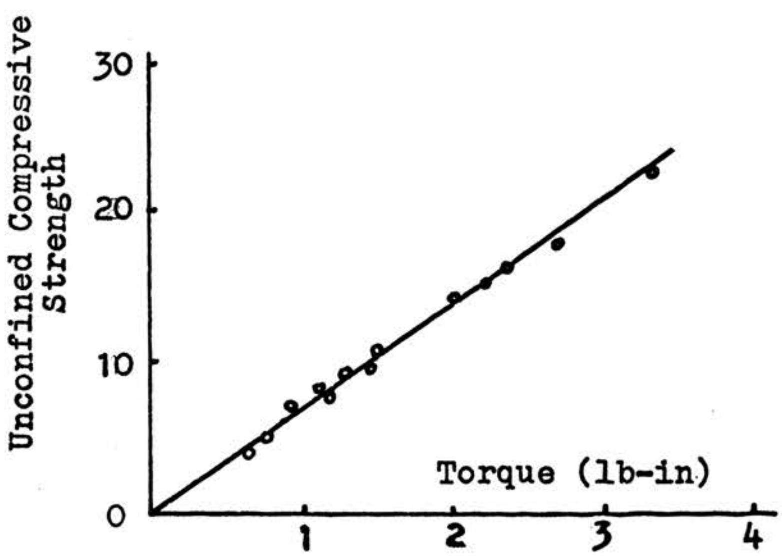
C = a constant

It was concluded from these experiments that the vane borer was a reliable piece of test equipment and could be used to measure the in situ shear strength of soils. It was further pointed out that the values measured near the surface of the ground compared favorably with those determined in the laboratory. However as the depth increases, the vane borer produces values which greatly exceed those determined by laboratory methods.

During this same period the British were also conducting experiments using the vane borer apparatus. Under the direction of I. Evans and G. G. Sherratt of the Army Operational Research Group, War Office, a Miniature Vane was developed for laboratory use. This smaller version of the vane borer is shown in Figure 4.[5]



a. Miniature Vane



b. Relation Of Torque To Compressive Strength

FIGURE 4.

The torque is applied through a worm and gear wheel mechanism by means of a crank. A spring calibrated for torque, is connected between the worm and gear wheel mechanism and the shaft of the vane. A pointer and protractor disc are mounted on a frame below the spring so that as the crank is turned the angular displacement is measured. This particular instrument has three interchangeable springs which are calibrated from 0.05 lb. in. to 4.4 lb. in. of torque.

Because of the small size of the laboratory vane shear device each of the four vanes was made 1/2 x 1/4 inch, also using a $\frac{H}{D}$ ratio = 2. To reduce fabrication problems and maintain mechanical stability the vanes and shaft were milled from a solid rod. The complete system was then mounted to a vertical column and threaded shaft which facilitated raising or lowering the entire mechanism by turning a second crank.

When conducting a test, the vanes were forced into the soil very slowly until the uppermost edges were one-half to one inch below the surface of the soil. A vane rotation rate of 1/60 r.p.m. was used when applying the torsional strain until the soil failed in plastic flow.

A fat clay from Pulborough, Sussex was tested in both the undisturbed and remolded state. Remolded tests were also run at varying moisture contents. The maximum angle displaced was converted to torque and these values were compared with the results of unconfined compression tests. By plotting torque as a function of compressive strength a calibration curve was established. Additional tests were also performed on a sandy clay and the experimental values compared very favorably with the theoretical values calculated. The soil used provided a range of ϕ angles of 3.0 to 14.0 degrees.

Hamilton Gray (1957, p. 844) further investigated the need for an apparatus to test the shear strength of soils in situ. Along with the many others who preceded him, he expresses the view that:

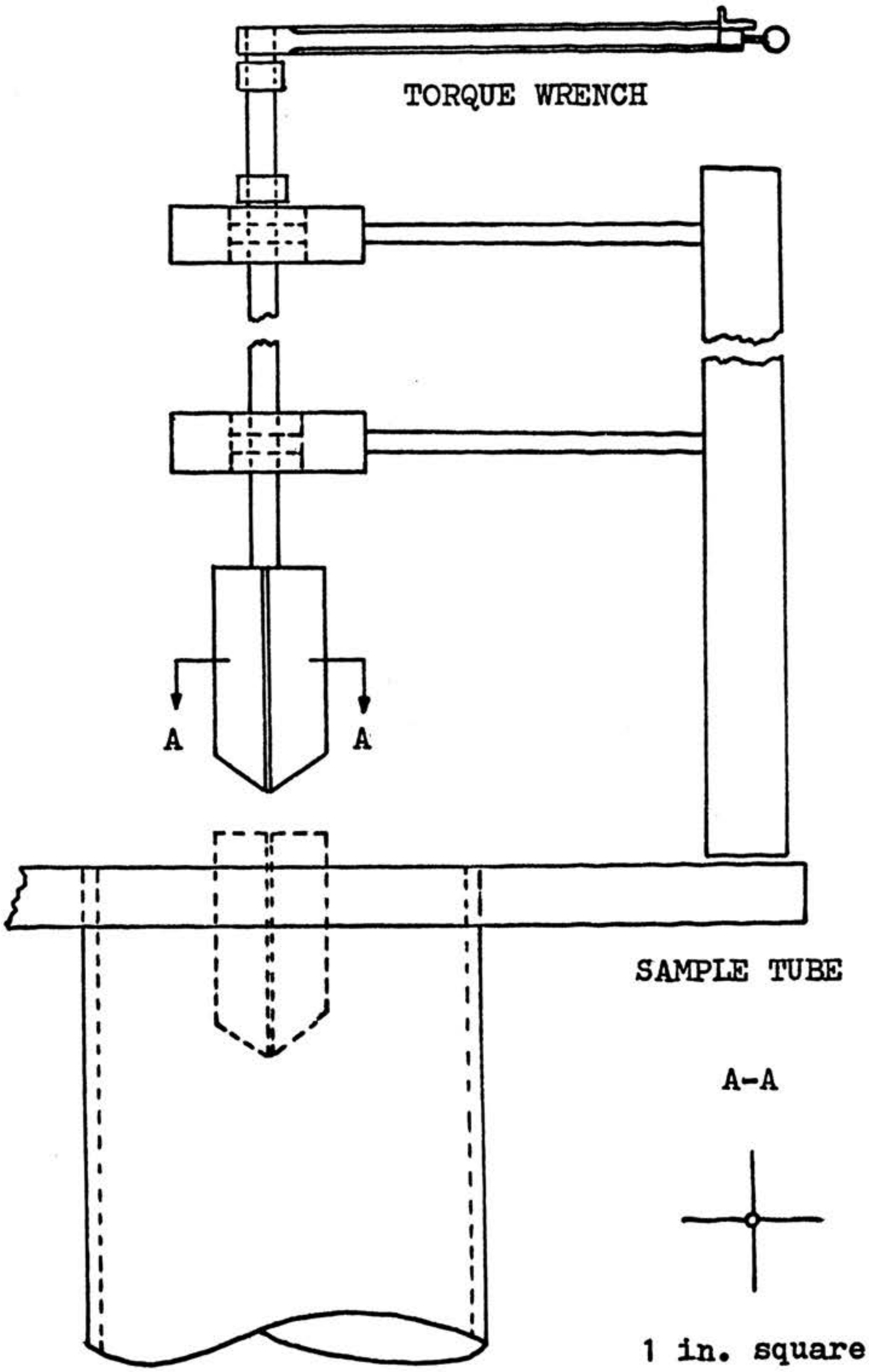
> "Despite successful efforts to develop techniques for minimizing the 'disturbance' to sensitive cohesive soils ... the best 'undisturbed' samples of many clays (also known as 'practical undisturbed samples') do not reveal with desirable accuracy the actual mechanical properties of the various soil layers from which they are extracted."(6)

His investigations included testing undisturbed samples taken with standard 3 1/2" O.D. shelby tubes. Unconfined compression tests were made on samples simply extruded from the shelby tubes, and on specimens which had been very carefully trimmed to a 2 inch square cross section. A comparison of the results indicated that very little difference existed between the values for most of the samples. He concluded that removal of the peripheral material made no significant change in the test results and that trimming the sample was an unnecessary and time-wasting refinement. Based upon these conclusions he constructed a small vane apparatus which could be used to test the undisturbed sample without removal from the sampling tube. A sketch of the essential features of this miniature vane shear apparatus is shown in Figure 5.(6)

Gray conducted additional investigations using a field vane on two different types of soil extending to depths up to 90 feet. Undisturbed samples taken from the same location were tested in the laboratory using the miniature vane and unconfined compression tests.

A comparison of values obtained from the three tests indicated the following:

1. The field vane test yielded the greatest shear strengths.
2. The laboratory vane results were intermediate.
3. The half unconfined compressive strength values were the smallest.



MINIATURE VANE SHEAR APPARATUS

FIGURE 5.

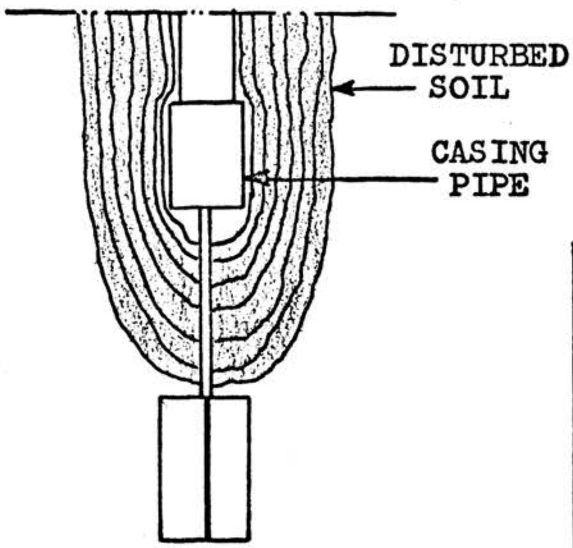
The results of this investigation by Gray showed that the vane strength is nearly always greater than half the unconfined compressive strength regardless of the care exercised in taking, preparing, and testing the undisturbed samples.

In his discussion concerning the results of Gray's investigation, Ebenezer Vey presents some additional factors which must be considered. Based upon his investigations Vey concludes that, "the actual shear resistance as measured by the vane then depends on: a) the diameter of the vane, b) the length of the vane, c) the stickiness of the soil, and d) the shear strength of the soil."(7) Vey determined this after performing a series of tests on remolded soils to determine the effects of friction between the soil and the vane blades. It was observed that a depression occurred around the vanes as they were inserted into the soil. Several clays of various consistencies were tested.

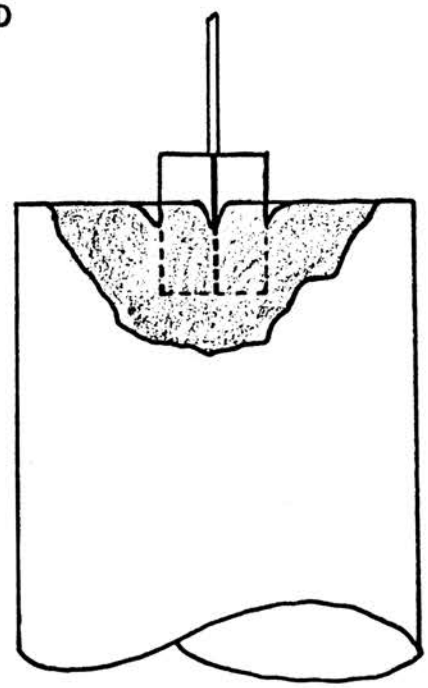
Cadling and Odenstad also considered this disturbance to the soil in their experiments. Sketches of this disturbance and depression are shown in Figure 6.(2) (6)

In June 1956 a symposium was conducted by the American Society for Testing Materials in Atlantic City, N. J. for the purpose of drawing the attention of the American soils engineers to recent developments and applications in the use of the vane shear apparatus. The results of investigations by such individuals as: Cadling and Odenstad, A. W. Skempton, Bennet and Mecham, Hamilton Gray and others were discussed. A report was published in 1957.(4)

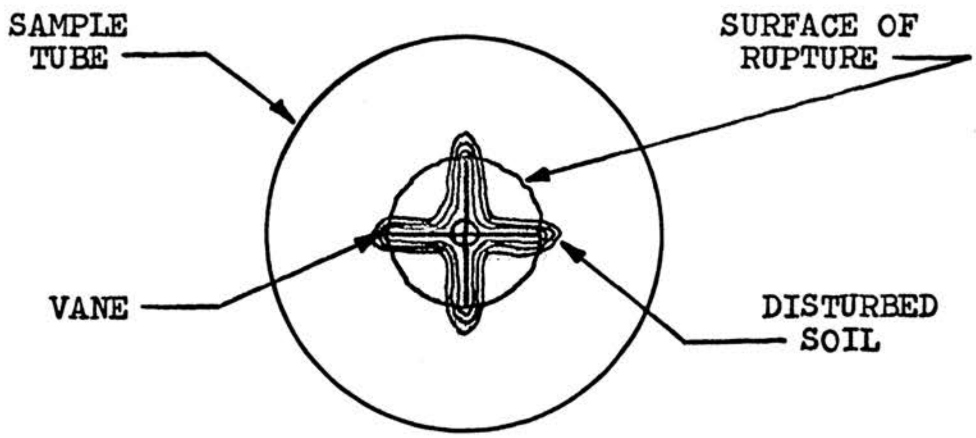
The U. S. Department of the Interior, Bureau of Reclamation has adopted the Inplace Vane Shear Test as a standard field test procedure. In the Bureau of Reclamation's "Earth Manual", this test is Des. E-20.



a. Disturbance Caused By Casing Pipe



b. Depression Caused By Vanes



c. Disturbance Caused By Vanes (Section Through Vanes)

DISTURBANCE OF SOIL AROUND VANES

FIGURE 6.

The apparatus is shown in Figure 7.[8] For vanes with an $\frac{H}{D}$ ratio = 2 the relationship of shear strength and measured torque is:

$$S = \frac{3T}{28\pi} r^3$$

Where: S = Shearing resistance of soil (psi)

T = Torque at failure (in.-lbs.)

r = Radius of vane (inches)

T. K. Liu and T. H. Thornburn (1963, p. 44) conducted a series of experiments to correlate the field vane strength with the unconfined compression strength of surficial soils. Due to the unsuitability of the unconfined compression test for this type of soil the test results were inconclusive.[9] However, as a result of regression analysis they were able to establish a relationship between the vane shear strength and the following soil properties: water content, liquid limit, plastic limit, plasticity index and liquidity index. From this they were able to estimate the field vane strength of a soil by the use of the expression:

$$C = 2.00 - 0.0415\,W + 0.0254\,PI$$

where: C = estimated field vane strength (Kips/Ft^2)

W = natural water content (%)

PI = plasticity index (%)

They concluded that the field vane strength of a surficial soil decreased with a corresponding increase in water content, and increased with an increase in plasticity. They also determined that the field vane strength was affected twice as much by a one percent change in water content as an equivalent change in plasticity.

In January 1964, the Waterways Experiment Station, U. S. Army Corps of Engineers published a report in which they compared the

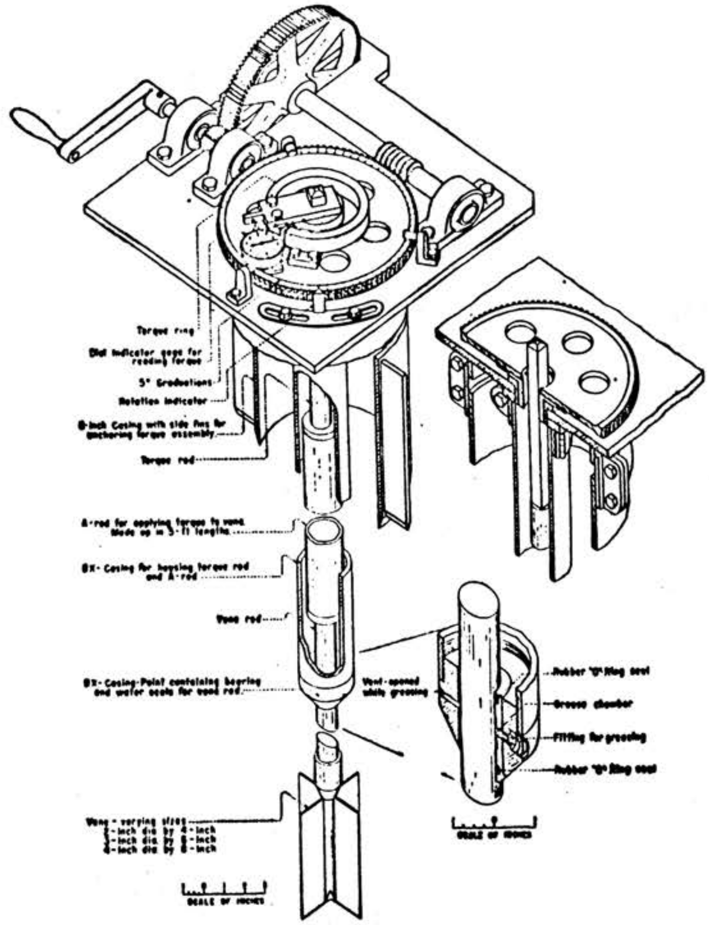


Figure 20-1.—Inplace vane shear test apparatus. 101-D-185.

INPLACE VANE SHEAR TEST APPARATUS

FIGURE 7.

results of testing a lean clay (CL) and a heavy (CH).[10] The purpose of this study was to investigate the strength-moisture-density relations of fine grained soils. The equipment used was the LLL Bevameter, the British shear vane and the WES cone penetrometer. The results of these tests were compared with the results of testing the same soils with the standard laboratory unconsolidated - undrained triaxial test.

A comparison of the vane shear strength and triaxial cohesion showed a reasonably good correlation for low values of shear strength and high degree of saturation. For lower degrees of saturation the results spread considerably. When the vane shear strength was compared with the triaxial shear strength a 1:1 relation existed over the entire range of soil conditions.

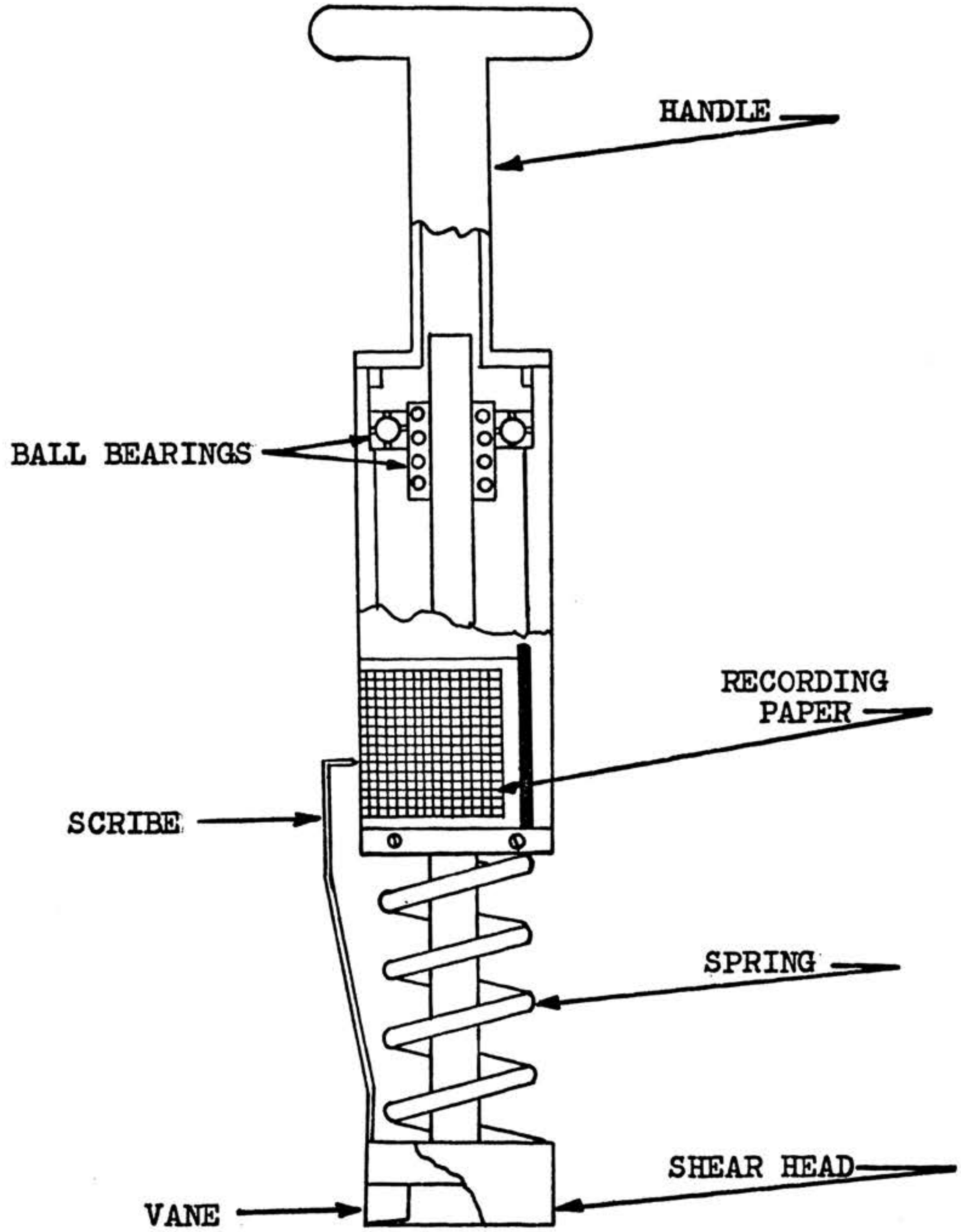
In 1952, P. C. J. Payne and E. R. Fountaine of the National Institute of Agricultural Engineering in England developed a torsional shear box which incorporated the principles of the shear vane and cylindrical shear box.[11] Their investigation was made in conjunction with the performance of cultivation implements and the shear strength of top soils in situ. The apparatus consisted of a shear head with six vanes, a torque meter and recording device and a series of slotted weights. When testing a soil the torsion box is forced into the soil and a predetermined number of weights are added to provide the necessary normal load. The torque is applied by twisting the handles until the soil fails by plastic flow. When testing fine-grained cohesive soils the test is an unconsolidated-undrained test as no time is allowed for drainage while the normal and shearing stresses are being applied. In the case of cohesionless soils this becomes a consolidated-drained test because drainage cannot be prevented.

The results obtained with this apparatus compare very favorably with the standard shear box, however, due to heavy weights and care required in preparing the sample the procedure is time consuming and tedious.

In 1962, Mr. G. T. Cohron modified this earlier model and developed the soil shear graph shown in Figure 8.[12] In place of the heavy lead weights a spring which was calibrated for both vertical and torsional displacement was used to measure the normal and torsional forces. In addition to adding the spring the size of the shear head was reduced and three vanes were used. A scribe mounted to the shear head etched the results of each test on pressure sensitive recording paper.

The soil shear graph yields good results when used on fine-grained cohesionless soils or cohesive soils which are nearly saturated. However, the reliability of these results decreases as the moisture content of the soil increases or when used on soil with a flocculated structure.[1]

It is hoped that this lengthy review of literature will assist the reader in understanding the previous studies which have been undertaken and prepare him for one more investigation of this problem.



SOIL SHEAR GRAPH

FIGURE 8.

III. GENERAL DISCUSSION OF THE PROBLEM

The shear strength of clay is usually determined on carefully selected specimens by trained technicians employing standard testing techniques. Studies on the shear strength of soils have been conducted by many private or semi-private organizations, governmental agencies and individuals. The sole intent of their research has been to find a method whereby the actual shear strength of soils could be determined in situ and on remolded laboratory samples. However, most of the equipment developed as a result of these studies is restricted to either field or laboratory tests. Studies are still being conducted to determine means of improving upon their reliability and accuracy. In conducting this investigation the primary purpose was to develop a compact portable piece of equipment which could be used in either the field or the laboratory.

The standard soil shear graph was selected as the basic mechanism because it is small, lightweight, compact and easily operated by one individual and has the capability of recording the results of tests performed. It was decided to use one type of soil, a clay of high plasticity, and to compact all samples with the same compactive effort at varying moisture contents. This soil is further described in Table I, Physical Properties of Red Plastic Clay.

Several modifications were considered including a detachable cutting edge to precede the existing shear head into the soil thereby reducing the effects of side friction. The second modification consisted of eliminating the circular cutting edge altogether and adding a shoulder to the shear head. The third and final modification was removal of the shear head and replacing it with a combination base plate and shear vane. Once these modifications were decided upon the test variables could be restricted to rate of strain application and the moisture content of the soil. A strain rate of 0.1 degree per second was selected based on the size of the vane being used. Moisture content was allowed to vary in order that a wide range of shear strength could be measured which would utilize the full capability of the shear graph.

The British laboratory vane shear and unconfined compression test were selected as standard tests for establishing a basis for comparing shear strength values.

IV. DESCRIPTION OF TEST APPARATUS, SOIL AND PROCEDURE

A. Test Apparatus

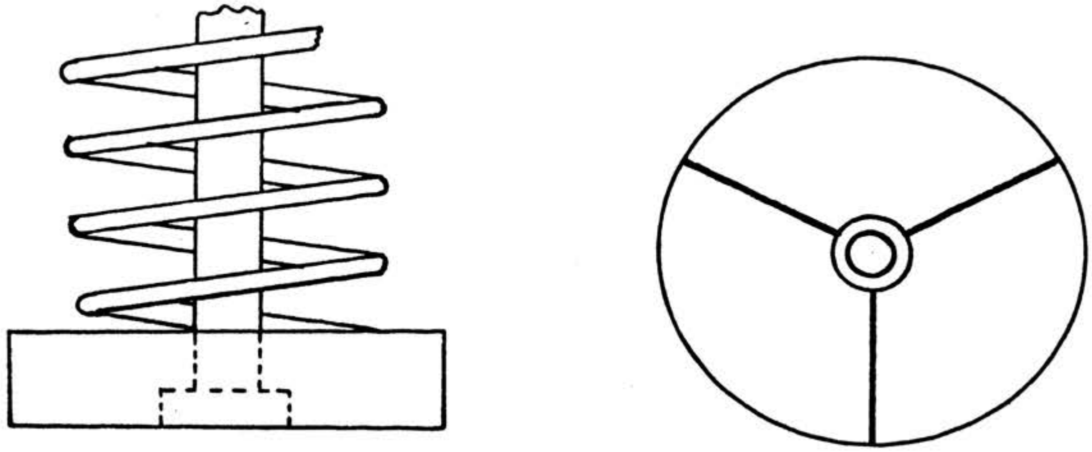
The basic test apparatus used in this investigation was the soil shear graph as shown in Figure 8. This is the standard soil shear graph as manufactured and sold by Soiltest, Inc.

The first modification consisted of removing the standard shear head and replacing it with one of five plexiglass heads, all of which were smooth and contained no cutting edge. Five heads were constructed varying in size from 1 19/32 inch outside diameter, the same size as the standard shear head, up to 3 19/32 inch outside diameter in 1/2 inch increments. The outside diameter of the vanes was held constant, and each head was machined to accept three vanes as shown in Figure 9a.

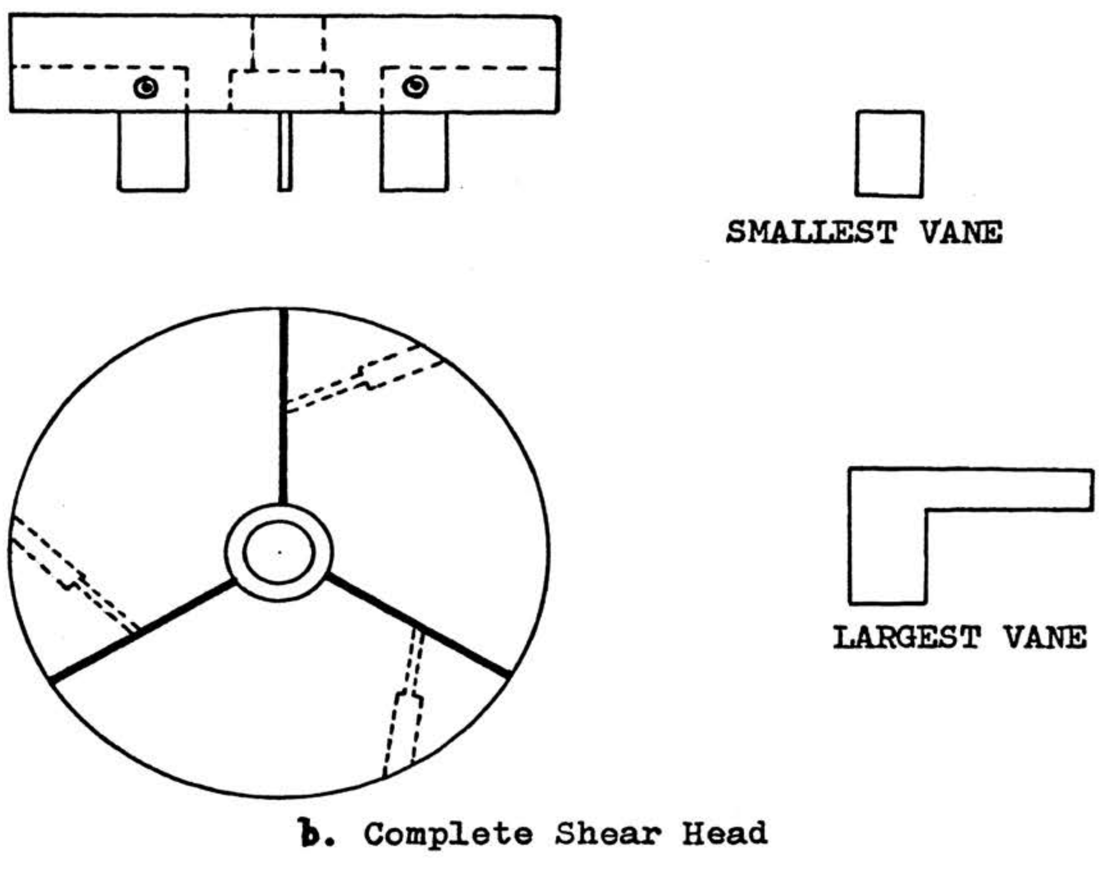
Five sets of vanes were cut from 0.028 inch stainless steel ranging from the standard height of 1/4 inch up to 1/2 inch by 1/16 inch increments. The width of the vanes was held constant at 7/16 inch. This allowed the various sets of vanes to be interchanged with the various size heads by merely loosening the three set screws in each head. The modified shear heads and shear vanes are shown in Figure 9b.

In order to insure that the surface of the shear heads would be smooth a plexiglass nut was used to secure the heads to the central shaft which acted as a stabilizer between the drum and the head being used. The torsion spring was secured to each head in the same manner as on the standard shear graph.

The pointer system functioned in the same manner as on the standard shear graph, however, this was later removed and a steel rod and



a. Shear Head



b. Complete Shear Head

MODIFIED SHEAR HEAD AND VANES

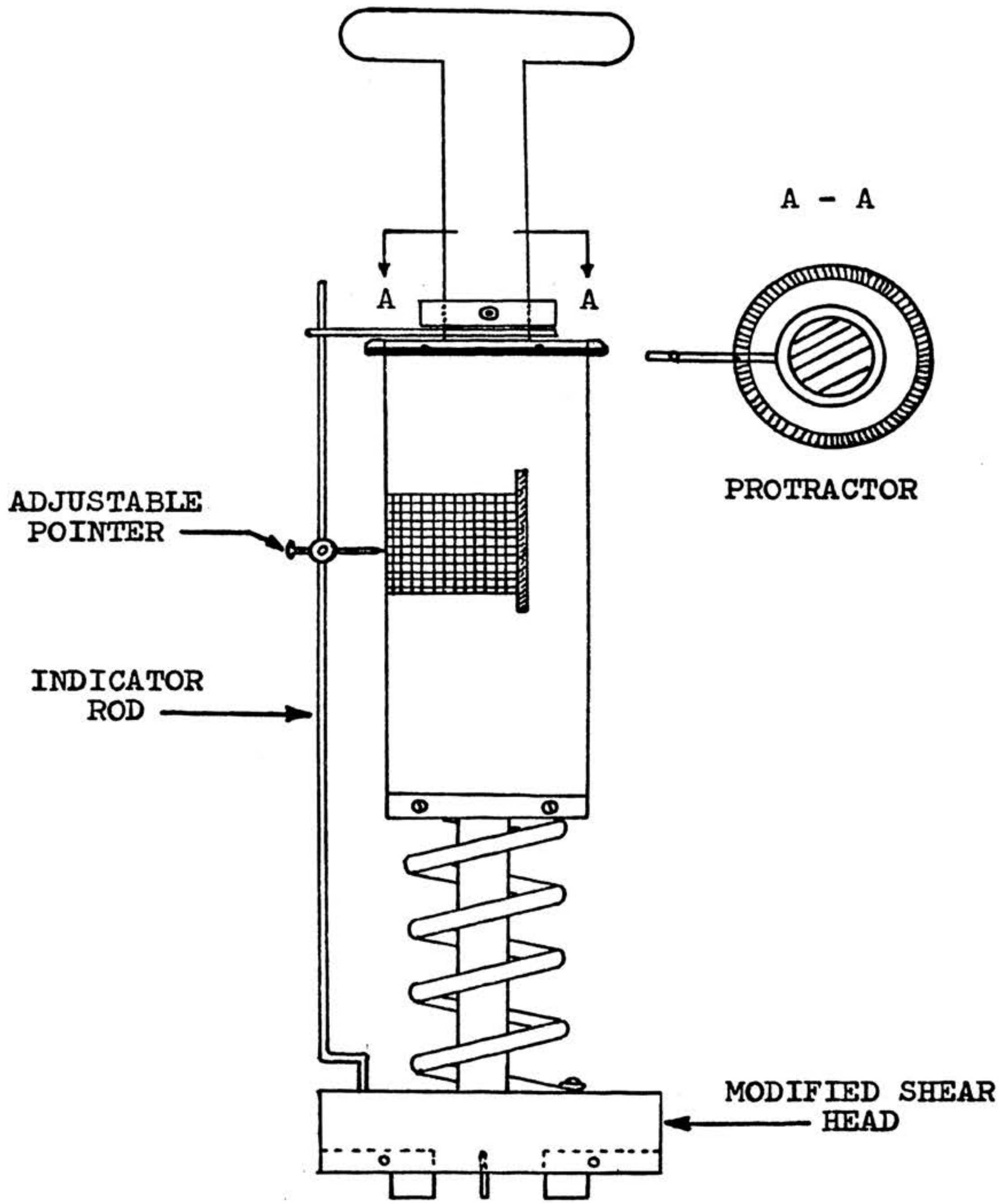
FIGURE 9.

protractor system was substituted so that the angle of rotation could be measured. This angle was then used to determine the torque required to cause a failure in the soil. Attached to the indicator rod was an adjustable pointer which was used to measure the vertical deflection of the spring. This pointer was secured to the indicator rod by a set screw and was used in conjunction with scales calibrated in psi of normal stress which were taped to the drum of the shear graph. This system is illustrated in Figure 10.

After several tests it became apparent that this system was not acceptable. The values of shear strength determined were too sporadic and usually much higher than the shear strength found from the tests being used for comparison.

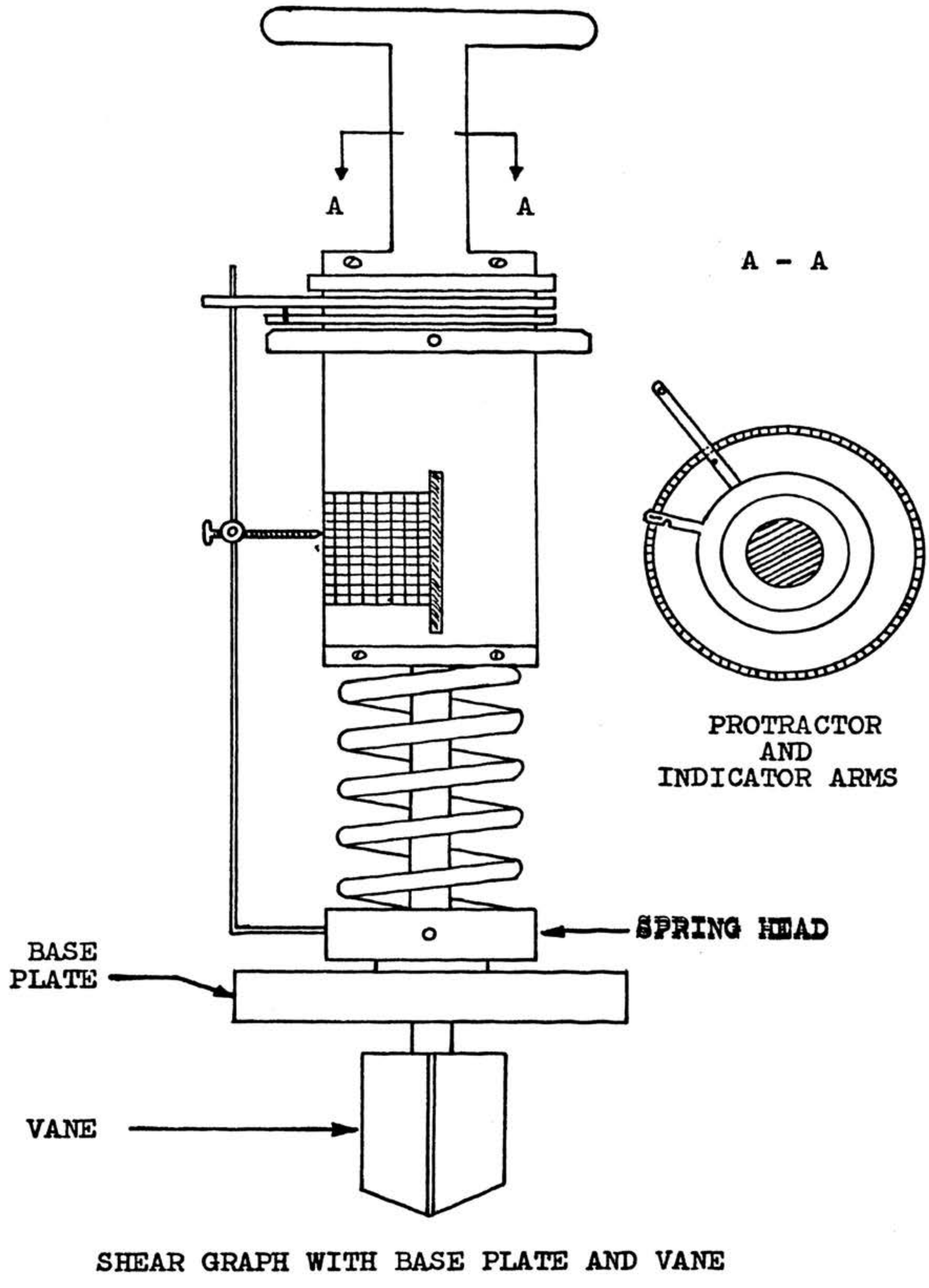
The final modification of the device is shown in Figure 11. In place of the shear head a base plate and vane mechanism were attached to the spring. The base plate was constructed to give a contact area of 10 sq. in. The results of preliminary tests indicated that a smaller contact area resulted in excessive penetration of the soil and caused preliminary failure upon application of a normal load. To reduce friction and allow the base plate to rotate freely, a standard generator ball bearing was used. The spring head was press-fit into the ball bearing which was in turn press-fit into the base plate. By keeping the tolerances small, it was possible to nearly seal the bearing between the spring head and the base plate preventing an accumulation of dirt and dust in the bearing. The details of the base plate attachment are shown in Figures 12 through 14.

By using a 1-inch square piece of brass stock it was possible to mill the vanes and turn the shaft as a single unit. The shaft was



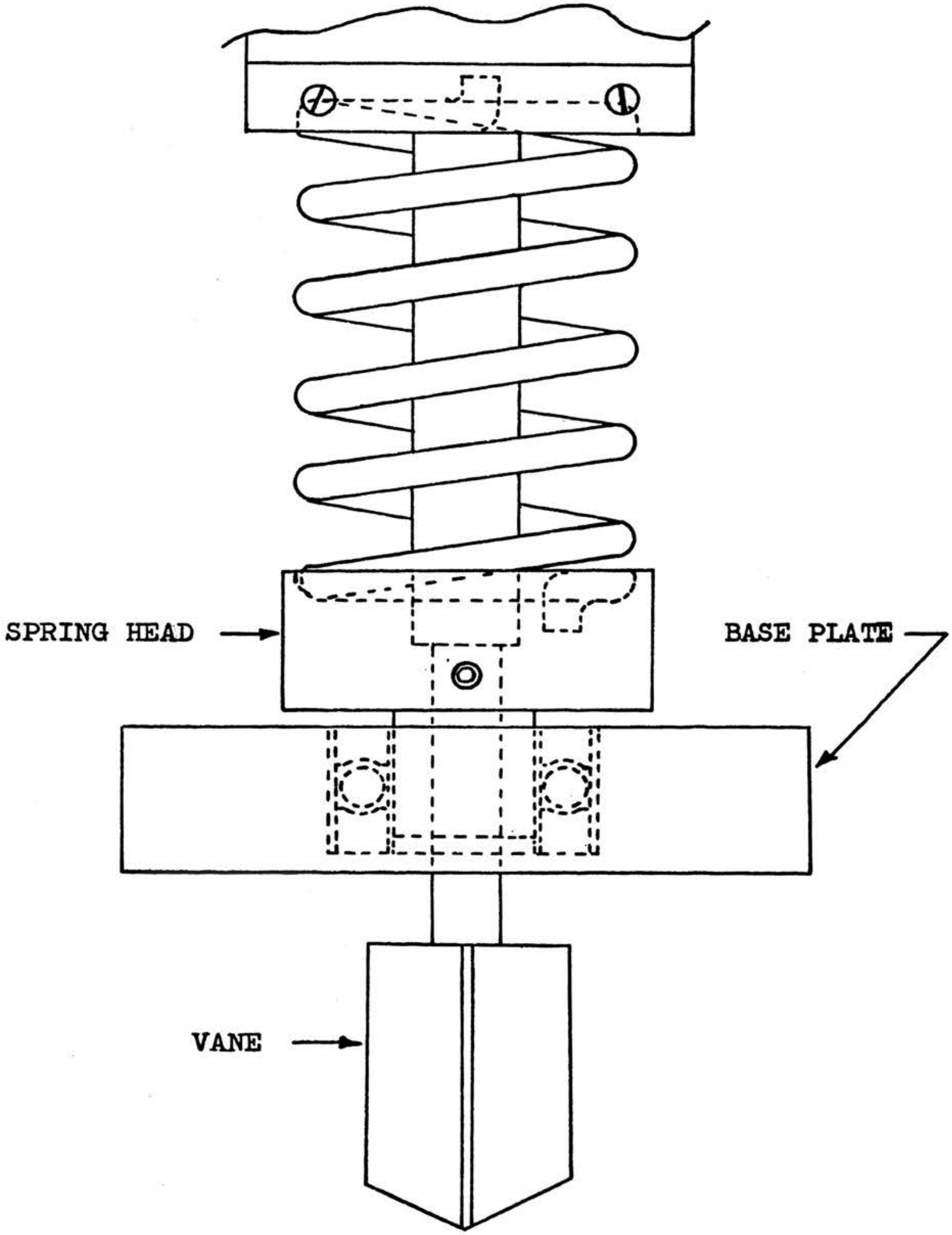
SHEAR GRAPH WITH MODIFIED SHEAR HEAD

FIGURE 10.



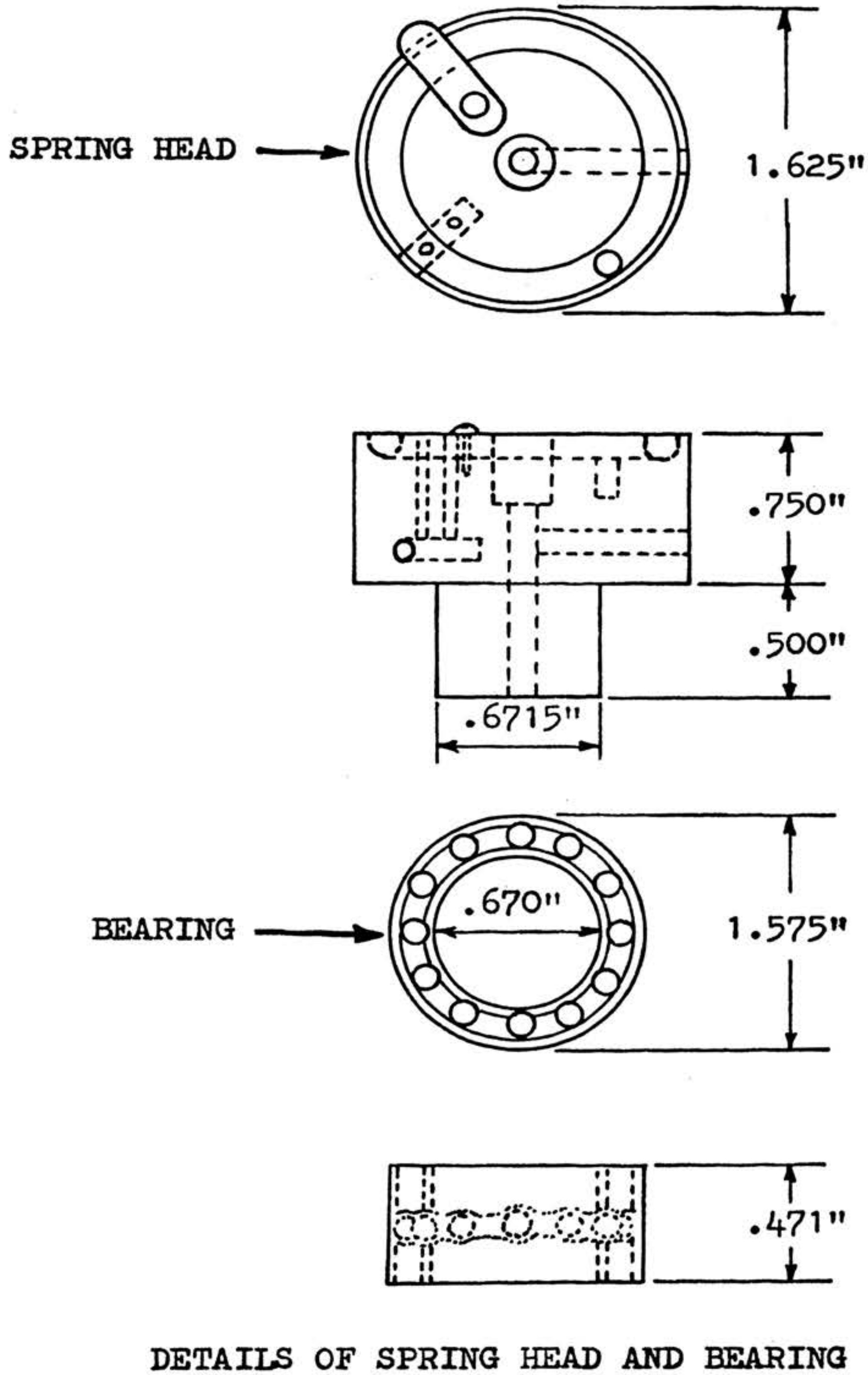
SHEAR GRAPH WITH BASE PLATE AND VANE

FIGURE 11.



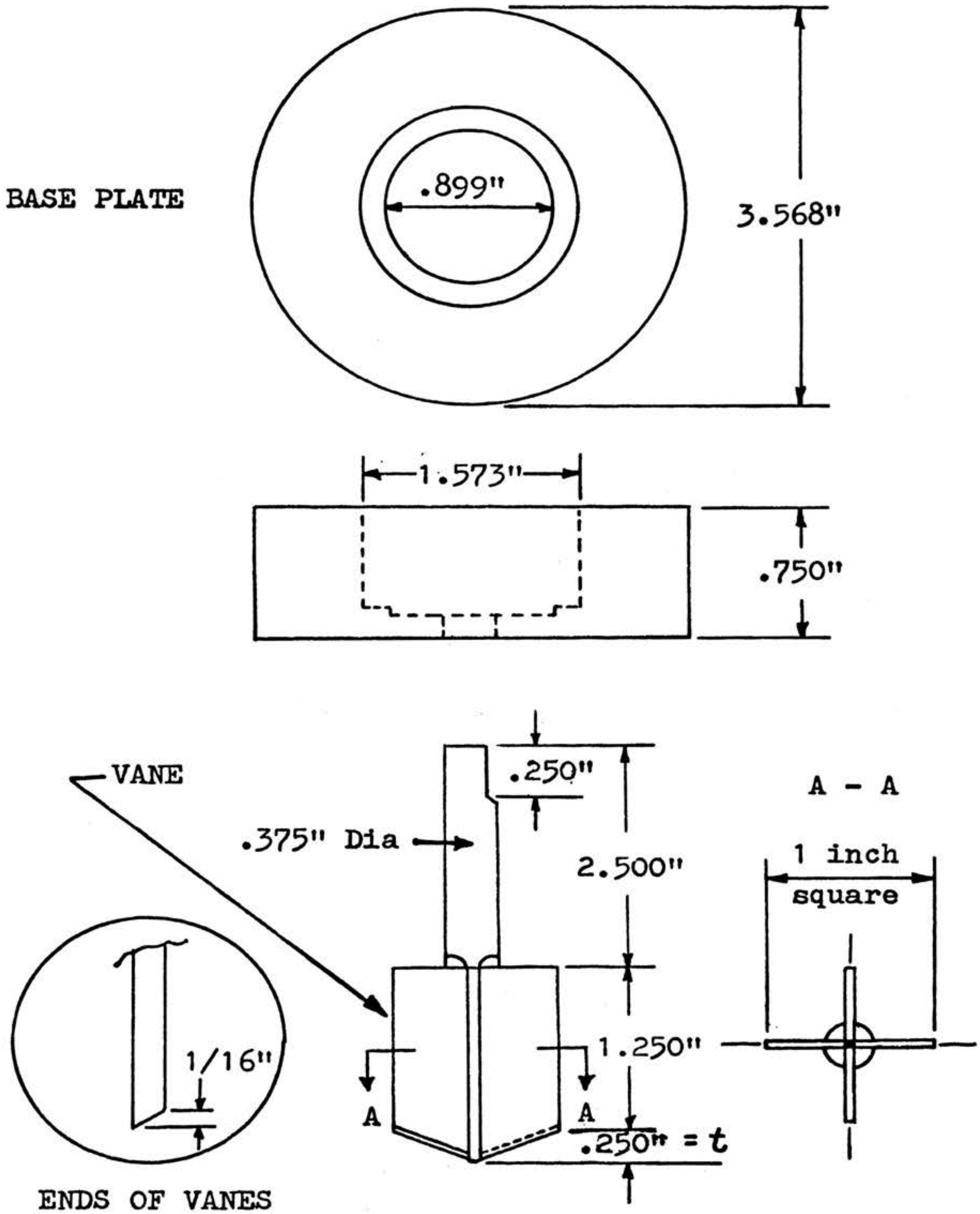
BASE PLATE AND VANE ATTACHMENT

FIGURE 12.



DETAILS OF SPRING HEAD AND BEARING

FIGURE 13.



DETAILS OF BASE PLATE AND VANE

FIGURE 14.

constructed to allow the top of the vanes to penetrate 1/16 inch below the surface of the soil. The vanes were milled to a thickness of 0.028 inches and the ends were bevelled to reduce disturbance when inserted into the soil. A height to diameter ratio of 1.25 to 1 was used for the vanes. The shaft of the vane is held in place by a set screw in the base plate to permit easy removal for cleaning. The theory of the modified shear graph is given in Appendix B.

B. Description of Soil

The soil used in this investigation was a red plastic clay obtained from the floor of Onyx Cave located approximately twenty miles southwest of Rolla. Onyx Cave is located in the Gasconade Formation which consists of a light brownish gray dolomite. Caves, sinks and springs are common in this area.

The floor of the cave consists mostly of this red plastic clay, however, a considerable amount of the soil is stratified with very fine and minute seams of silty material. Careful selection was imperative in order to obtain samples of pure red clay to be used for the investigation. The physical properties of this soil are listed in Table I.

TABLE I. PHYSICAL PROPERTIES OF RED PLASTIC CLAY

Field Moisture Content	64.6
Optimum Moisture Content	32.0
Liquid Limit	73.0
Plastic Limit	25.6
Plasticity Index	47.4
Shrinkage Limit	14.2
Specific Gravity of Solids	2.67
Sensitivity	4-8
Unified Classification	CH
B.P.R. Classification	A - 7 - 6
Primary Constituent	Illite
Secondary Constituent	Chlorite

The tabulated values were obtained from Atterberg Limits tests, specific gravity, standard proctor density tests performed according to the specifications of the American Society for Testing Materials, ASTM, Procedure for Testing Soils.

C. Test Procedure

Soil used for the remolded tests was carefully selected and inspected to insure that there were no silt and sand lenses present in the samples. Moisture content samples were taken at various points in the area from which the soil was removed. These samples were then used to determine the field moisture content of the soil in situ.

After removal of sufficient soil from the floor of the cave it was immediately transported to the laboratory. This soil was placed in a metal container and stored in the moist room until it was removed for testing.

Each sample used for the remolded tests was kneaded by hand and mixed with a trowel to insure the structure had been completely broken down. The samples were then compacted in standard four inch proctor molds using either the motorized compaction machine or the hand dropped, 5.5 lb., hammer. In all cases the soil was placed in the molds in five equal layers and compacted by 25 blows per layer.

Following completion of the compaction procedure the mold was cleaned, leveled and the density determined. Once this had been accomplished tests were conducted using the shear graph, the British vane shear and the modified shear graph.

In order to obtain as many tests as possible per sample, three tests were performed on both surfaces of the sample by each testing device.

After completion of these tests the soil was ejected from the mold and a 1-inch square x 2-inch long specimen was carved from the sample for testing in unconfined compression. Moisture content samples were taken from the excess soil removed while carving the specimen. This procedure was repeated until there was insufficient soil left to perform another test or the moisture content had decreased to the point where the indicated shear strength was greater than could be recorded by the soil shear graph, in this case 30 psi. When this occurred a new sample was removed from the moist room and the entire procedure repeated.

Undisturbed samples were obtained by using 3 inch thin walled Shelby tubes having an inside diameter of 2 7/8 inch. The tubes were 30 inches long with the edges sharpened and tapered to reduce disturbance of the soil entering the sampler. The tubes were pushed into the soil by hand and then rotated several times before being extracted. After all tubes had been filled they were immediately transported to the laboratory and stored in the moist room. The tubes were covered with damp burlap to preserve the natural field moisture content until removal for testing.

These undisturbed samples were then removed from the moist room and placed in the extruder. After a portion of the sample had been extruded it was trimmed flush with the end of the tube to give a smooth testing surface. One test was made with the shear graph and the modified shear graph with the vane attachment to obtain the indicated shear strength. Two tests were performed with the British vane shear. After completion of these tests the sample was extruded further and carefully trimmed for unconfined compression test testing. Moisture content samples were taken from the soil removed during the trimming process.

Fifty tests were performed with the modified shear graph throughout the area where the undisturbed samples were removed. A moisture content sample was taken from the soil around each shear strength test.

The test results of all tests performed are summarized in the figures of moisture content or dry density versus shear strength for the various types of test equipment used, and the figures of shear strength as indicated by the modified shear graph versus the British vane shear, shear graph and unconfined compression test. These data are tabulated in Appendix C.

V. TEST RESULTS

A. Introduction

To obtain an insight into the complexities of shearing resistance of clay soils certain properties must be considered. Some of these properties are, (a) moisture content, (b) sensitivity, (c) consistency, (d) mineralogy, (e) method of deposition and (f) amount and type of preconsolidation load. Probably the two most important properties are consistency and sensitivity.

Consistency is that property of cohesive soils which offers resistance to external forces tending to rupture or deform the soil aggregate. Consistency is a function of cohesion (or adhesion) of the clay particles which in turn is a function of moisture content. Unlike cohesionless soils, which depend upon intergranular friction for their strength, pure cohesive soils obtain the bulk of their strength from the attraction forces of the minute water films surrounding the particles. Consistency is expressed as a function of the liquid limit in situ moisture content and plasticity index. Common terms used to express consistency are, very soft, soft, firm, stiff or hard. For correlation purposes consistency is usually compared with the undisturbed strength. A typical comparison is shown in Table II[13].

As was stated above, the sensitivity is also important in determining the shear strength of cohesive soils. Sensitivity is defined as the ratio of the undisturbed strength to remolded strength at the same moisture content. The sensitivity varies for different clays and will also vary for the same clay at different moisture contents. Some clays exhibit such high sensitivities that they turn into viscous fluids upon remolding. Methods of expressing sensitivity are shown in Table III.[14]

The in situ moisture content of the red plastic clay is very close to saturation, therefore, if 100% saturation is assumed the Ø angle will equal zero. From these assumptions it follows that any increase in maximum principle stress will be taken up by the pore water pressure and there will be no change in the effective stress. The very low permeability of the clay results in practically no drainage of the specimen during testing. Because no drainage occurs there is no resulting consolidation or increase in effective stress in the sample and the test then becomes an unconsolidated-undrained or quick test.

All of the test methods used are basically similar. The unconfined compression test, modified shear graph and standard shear graph have the capability for applying varying normal loads. In the case of the laboratory vane shear the normal load is a function of the overburden pressure of the soil and varies only with depth. The only difference between the various testing methods is that the test specimen subjected to the unconfined compression test has been removed from the sampling tube or proctor mold. However, due to the geometric design of the vanes and method by which the load is applied, the failure plane is vertical and confinement does not affect the test procedure. In addition, the permeability of the clay is very low and no drainage occurs during shear in either method.

From the assumptions made at the beginning and the principles of the test procedures used, it can be shown that the shear strength is a function of the cohesion of the soil and the normal load on the plane of failure. Therefore, since all of the test methods selected produce

TABLE II

CONSISTENCY IN TERMS OF UNCONFINED COMPRESSIVE STRENGTH[13]

Consistency	Unconfined Compressive Strength	
	Kg/Sq Cm	Lbs/Sq in.
Very Soft	< 0.25	< 3.55
Soft	0.25 - 0.5	3.55 - 7.11
Medium	0.5 - 1.0	7.11 -14.22
Stiff	1.0 - 2.0	14.22 -28.44
Very Stiff	2.0 - 4.0	28.44 -56.88
Hard	> 4.0	> 56.88

TABLE III

CLASSIFICATION OF SENSITIVITY[14]

Sensitivity (S_t)	Classification
1	Insensitive
1 - 2	Slightly Sensitive
2 - 4	Medium Sensitive
4 - 8	Very Sensitive
4 - 16	Slightly Quick
16 - 32	Medium Quick
32 - 64	Very Quick
64 - inf	*Extra Quick

*Extra Quick - Viscous Liquid

the same type of test, the results from any one method can be compared with the other three.

B. Undisturbed Test Results

The natural moisture content of the red plastic clay used in this investigation ranged from 60 to 80 percent with a liquid limit of 73 percent, and a plastic limit of 25.6 percent. Figure 15 shows that within this range of natural moisture contents, the undisturbed shear strength varied from a low of 1.4 psi to a high of 9.1 psi. A comparison of values of shear strength as indicated by the four pieces of test equipment reveals that (a) the unconfined compression tests yielded the lowest values, (b) the laboratory vane shear intermediate values, (c) the modified shear graph the highest values and (d) the shear graph values were spread throughout the range covered by the other three types of tests.

There are two factors which influenced the lower values yielded by the unconfined test. Even though extreme care was used in preparing the samples for testing it was impossible to obtain a truly undisturbed sample because of the high sensitivity of the clay. The sensitivity was found to range between 4 and 8, with a consistency ranging from very soft to medium. Several samples were discarded because of the excessive deformation that occurred before the load could be applied. The second factor which caused these lower values was the adverse influence of very minute seams of silty material. Although the surface of the strata appeared to be free of these intrusions it was apparent after extrusion from the sampling tubes that these thin seams existed throughout the deposit. To reduce this effect to a minimum, the samples were trimmed such that these seams were as nearly horizontal

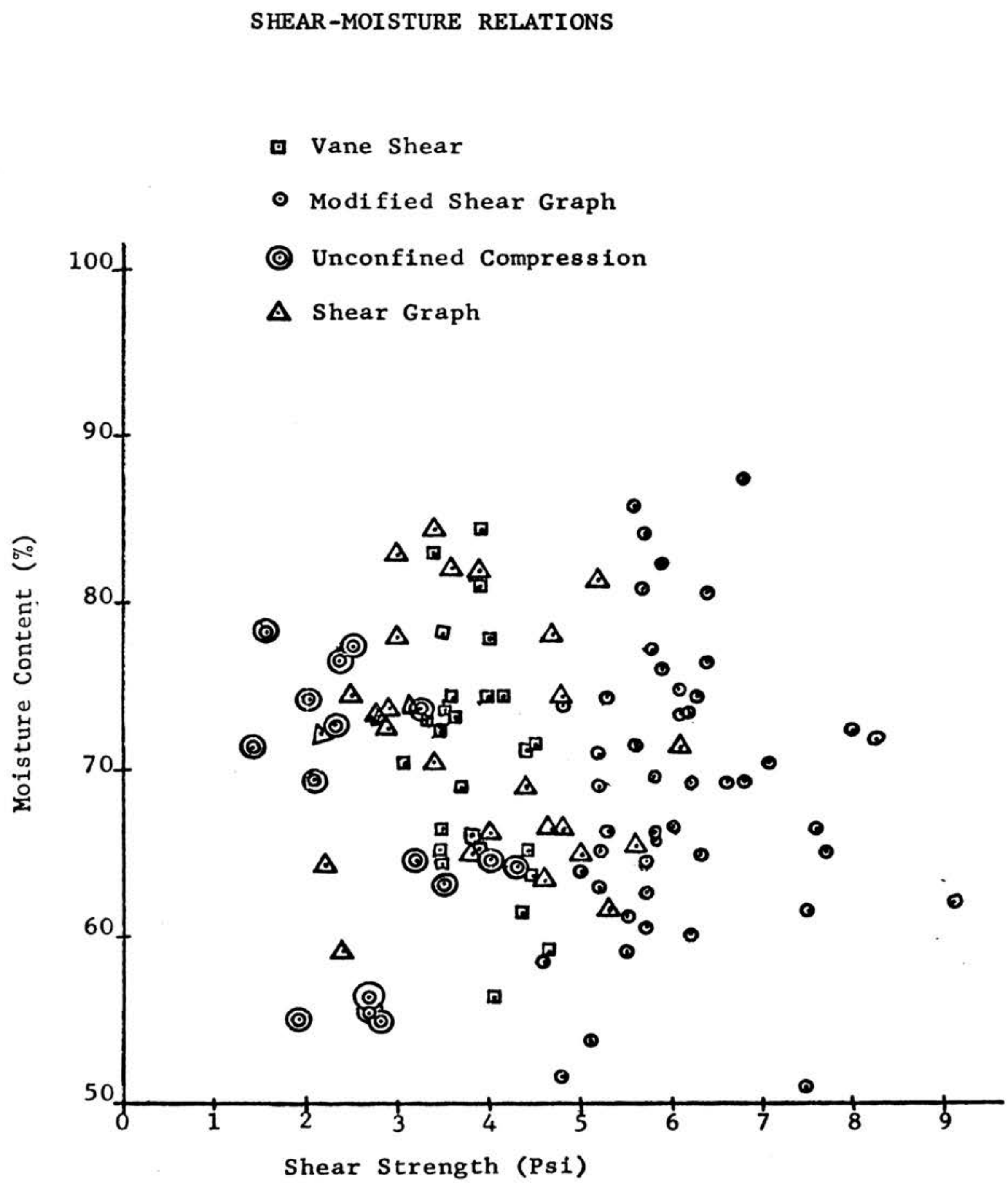


FIGURE 15. UNDISTURBED SOIL

as possible. These seams did not affect the other three tests due to the geometric design of the vanes which held the soil between the wings as the vane was rotated.

A comparison of the values obtained with the modified shear graph, the shear graph, the laboratory vane and the unconfined compression test are shown in Figures 16, 17 and 18 respectively. Because of the range of values for the shear strength at any one moisture content, no direct correlation between the modified shear graph and the other methods used can be made.

C. Remolded Test Results

When the natural moisture content of a cohesive soil is near the liquid limit and the soil is medium to highly sensitive, the undisturbed shear strength may be relatively high. However, at the same moisture content remolded samples of the same soil will have very little shear strength.

It has been shown that the natural moisture content of this clay varies within ± 7 percent of the liquid limit and exhibits a high sensitivity. Accordingly then, the remolded shear strength should be very low when compared to the undisturbed shear strength at the same moisture content. This is verified by the shear-moisture relation shown in Figure 19. Values ranged from 0.47 psi to 1.32 psi for the remolded clay as compared with 1.4 psi to 9.1 psi for the undisturbed clay. Figure 20 shows the relationship between remolded shear strength and dry density.

Figures 19 and 20 indicate that a logarithmic relationship exists between the shear strength, moisture content and dry density of the remolded clay. The natural moisture content of the clay is in excess

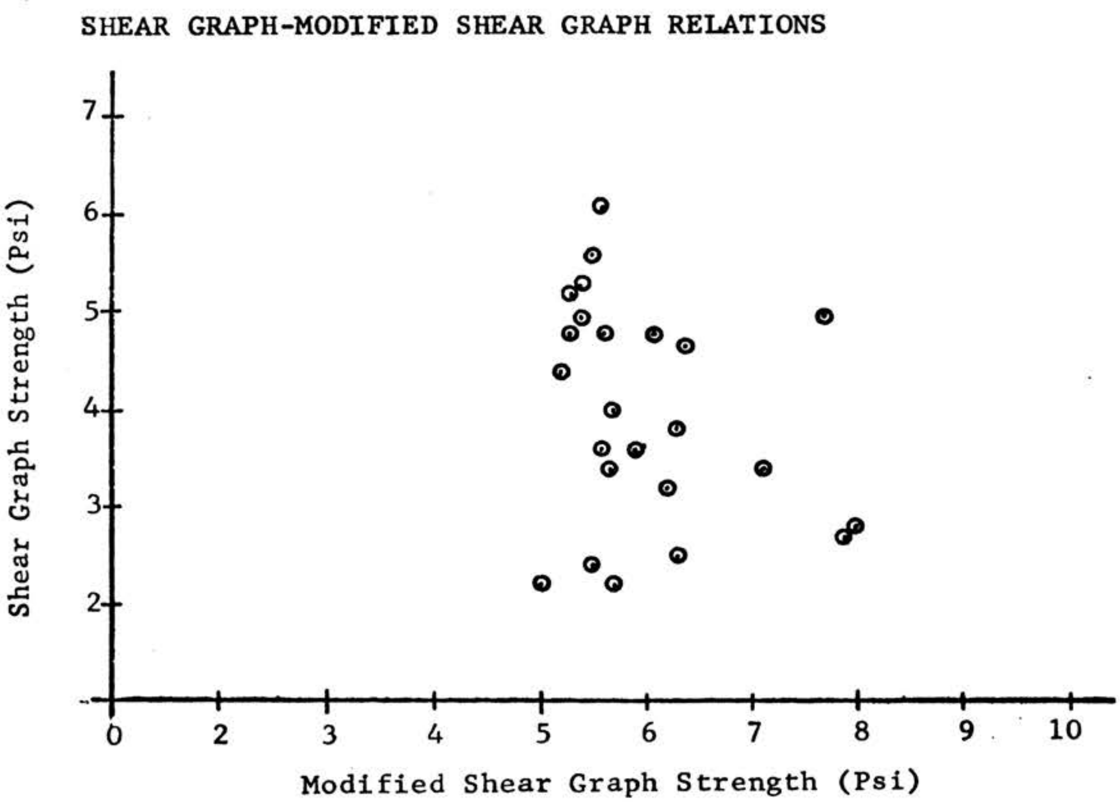


FIGURE 16. UNDISTURBED SOIL

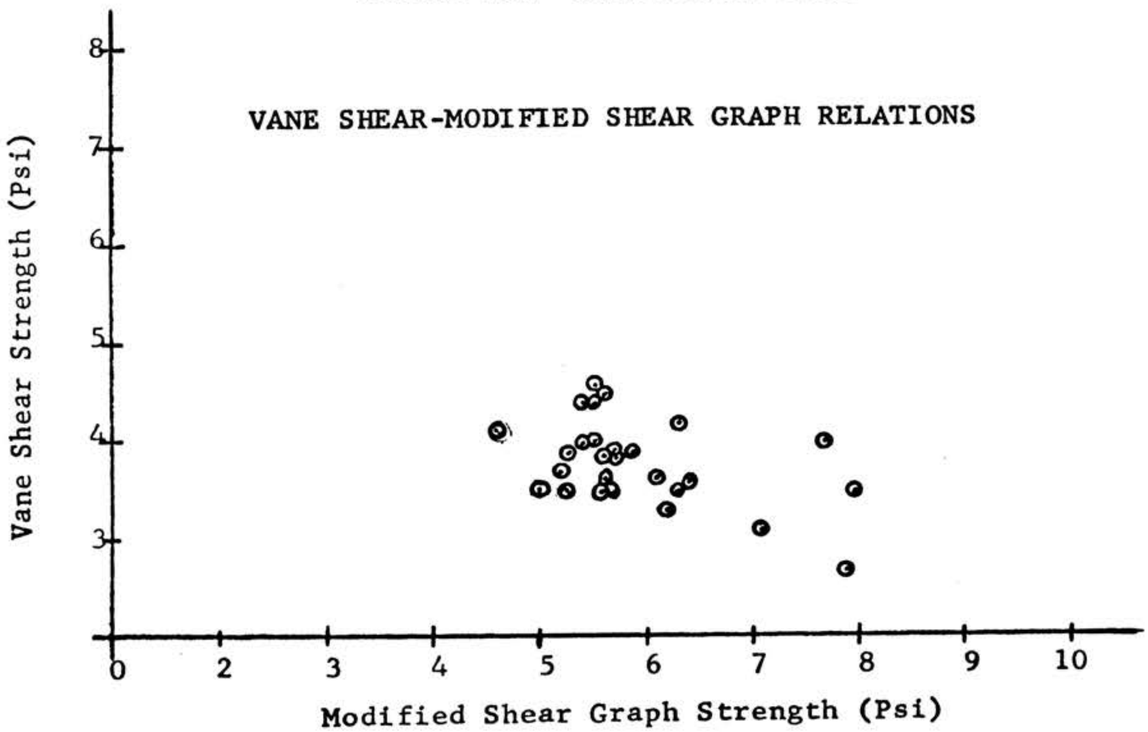


FIGURE 17. UNDISTURBED SOIL

UNCONFINED-MODIFIED SHEAR GRAPH RELATIONS

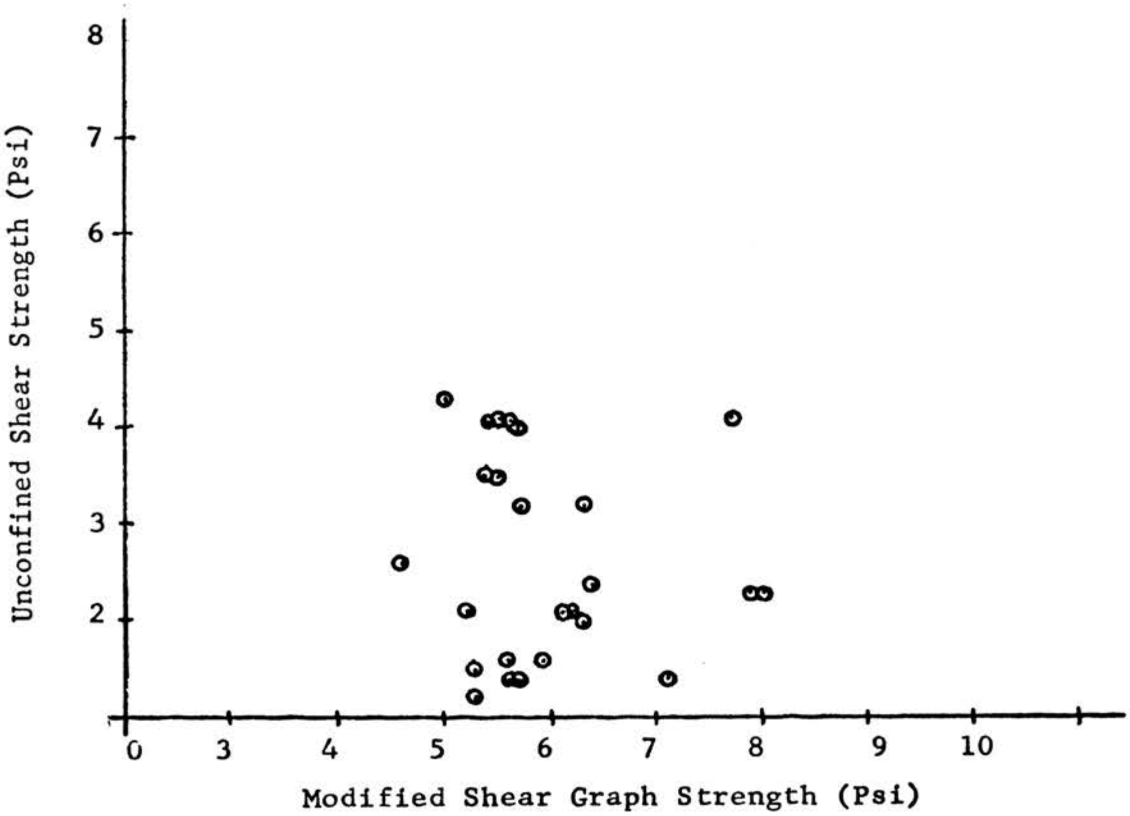


FIGURE 18. UNDISTURBED SOIL

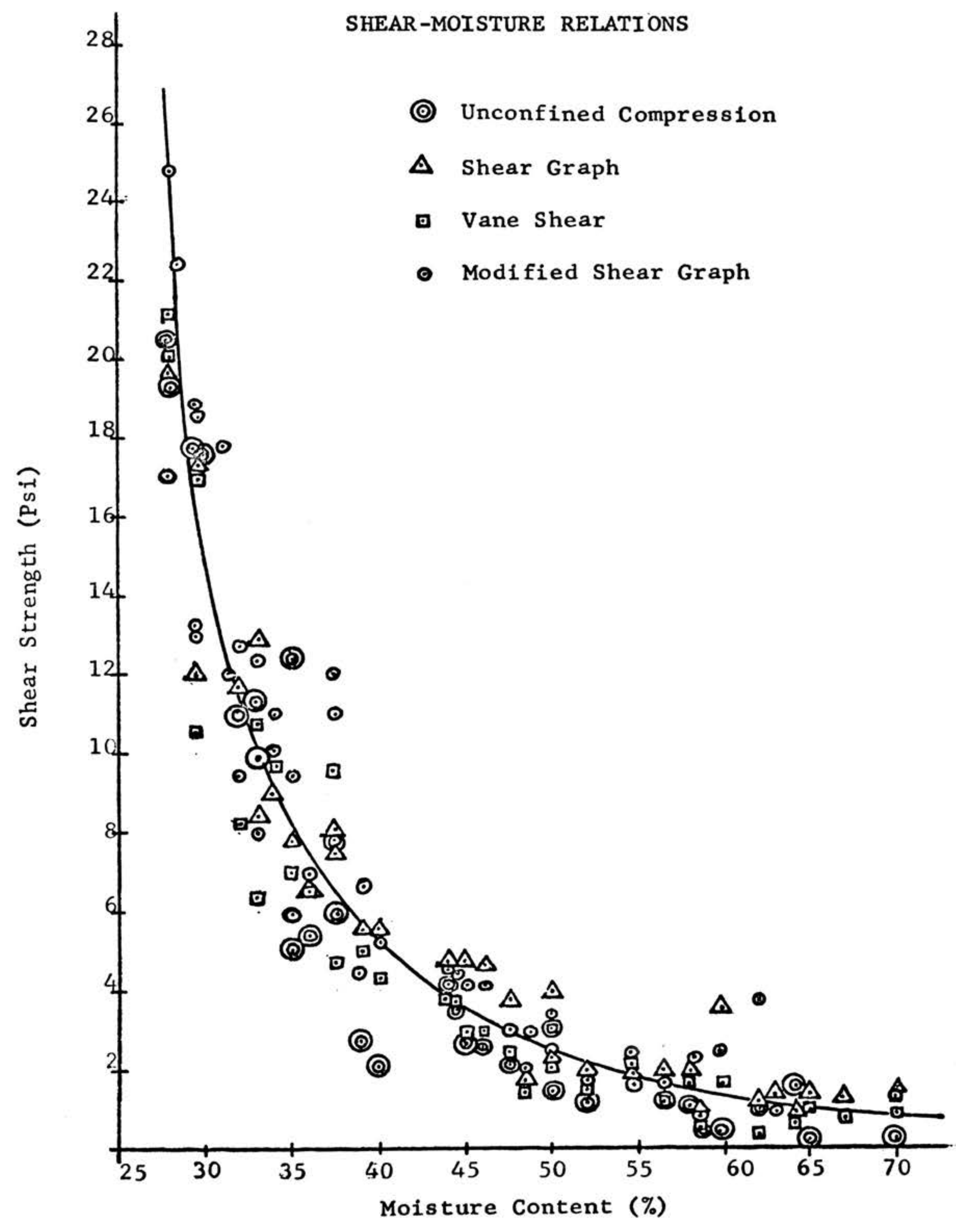


FIGURE 19. REMOLDED SOIL

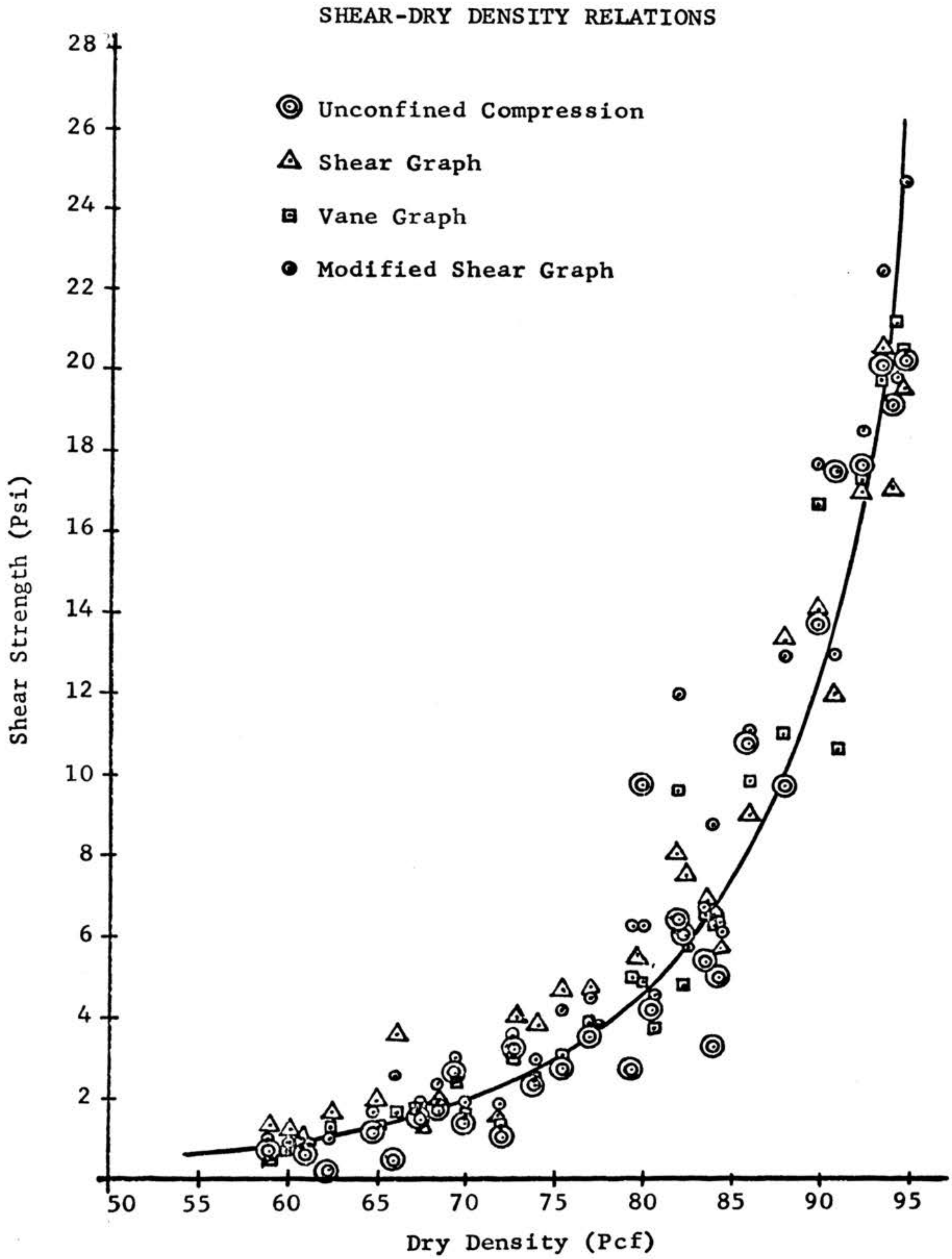


FIGURE 20. REMOLDED SOIL

of optimum, and as a result any increase in moisture content will cause a decrease in dry density and a decrease in the resulting shear strength. This fundamental relationship for cohesive soils is verified by all of the test results.

Both the shear-moisture plot and the shear-density plot, indicate that (a) the shear graph yielded the highest values, (b) the unconfined compression tests the lowest and, (c) the modified shear graph and laboratory vane shear intermediate values. As a general rule the modified shear graph and laboratory vane shear matched very closely. This can be attributed to the similarities of the two instrument including (a) type of load applied to the soil, (b) method of application, (c) rate of strain induced and (d) human factor. Both of these devices measure the shear stress developed in vertical planes as a result of an applied torque. The torque is transferred through a spring to the vanes which in turn applies the shear force to the soil. The applied rate of strain was the same for both instruments, i.e., 0.1 degrees of rotation per second.

The modified shear graph strength versus moisture content as shown in Figure 21 indicates that there exists a fairly close relation except between the range of 30 percent and 40 percent moisture. This is also true of the modified shear graph-dry density plot as shown in Figure 22. Between 80 pcf and 90 pcf dry density the values of shear strength vary to a greater degree than in any other range of dry density.

The optimum moisture content of 32 percent for standard compactive effort lies within this range of moisture content. It has been shown that the optimum moisture content for the maximum compressive strength is always less than the optimum moisture content for maximum dry

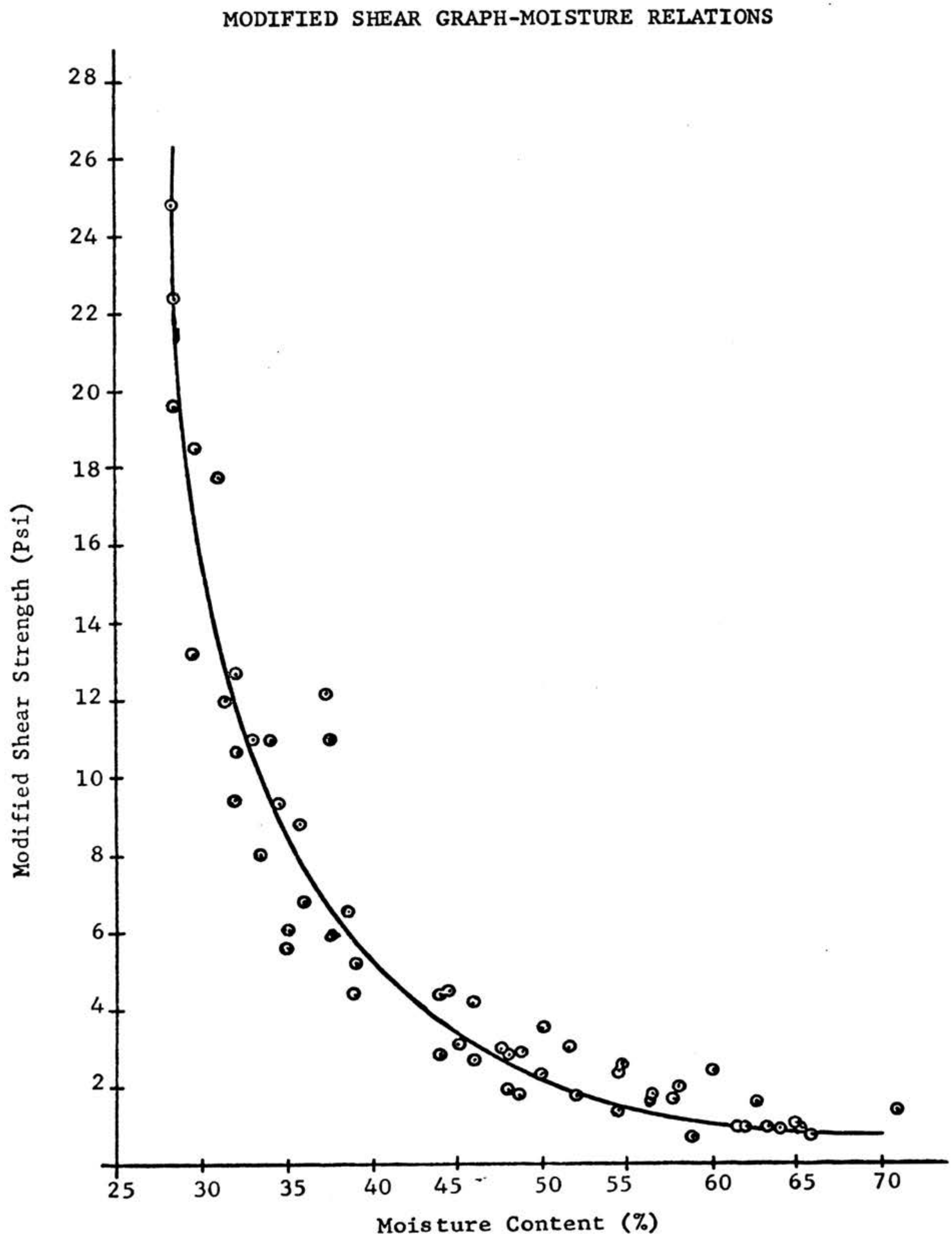


FIGURE 21. REMOLDED SOIL

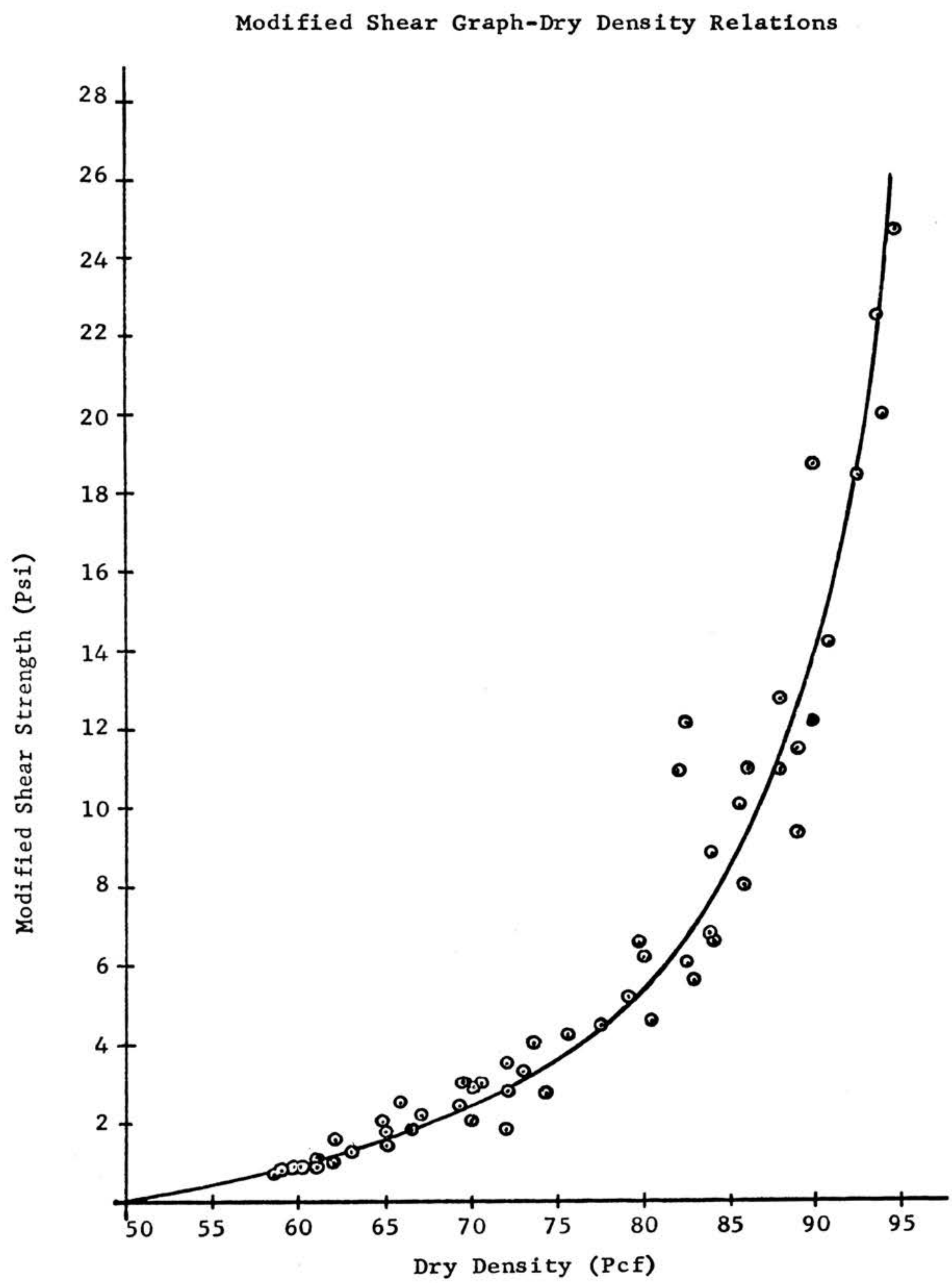


FIGURE 22. REMOLDED SOIL

density.[15] As the moisture content increases beyond the optimum for compressive strength there is a decrease in strength while the dry density continues to increase. At this point the additional moisture is absorbed by the clay particles causing them to become dispersed. This change in structure is a result of particle reorientation which in turn causes a decrease in the shear strength of the clay. This fact would explain the scattered results of indicated shear strength within this range of moisture contents and dry density. Additional errors or variances can be attributed to the sensitivity of the equipment to include the springs, rate of strain applied and human elements.

In Figures 23, 24 and 25, the modified shear graph strengths are plotted as a function of the shear strengths as measured by the laboratory vane shear, shear graph and unconfined compression tests. As has been shown in Figures 19, 20, 21, and 22, between 30 percent and 40 percent moisture content or 80 pcf and 90 pcf dry density, the greatest spread in the values of shear strength occur. In Figure 23, modified shear graph versus laboratory vane shear, this spread does not appear to be as great. The reasons for this can be directly related to the similarities between the two pieces of test equipment. This close relationship strengthens the comparability characteristics of the modified shear graph and laboratory vane shear for testing the shear strength of cohesive soil.

A comparison of values indicates that the modified shear graph yields values 1.167 times the laboratory vane shear and 1.235 times the shear graph. A complete comparison with the unconfined compression test is not possible however, because the degree of spread is much greater than with either of the other two pieces of test equipment.

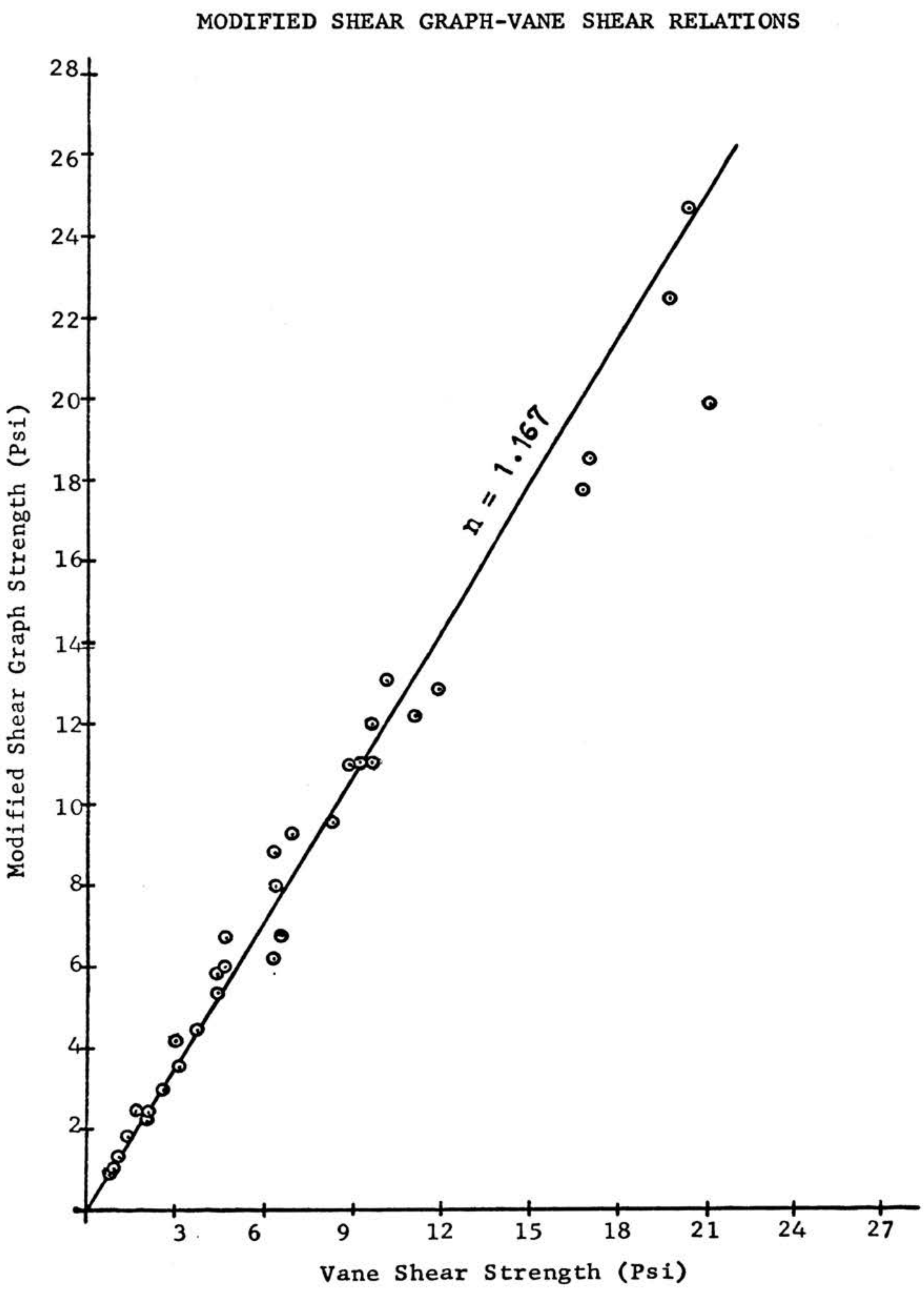


FIGURE 23. REMOLDED SOIL

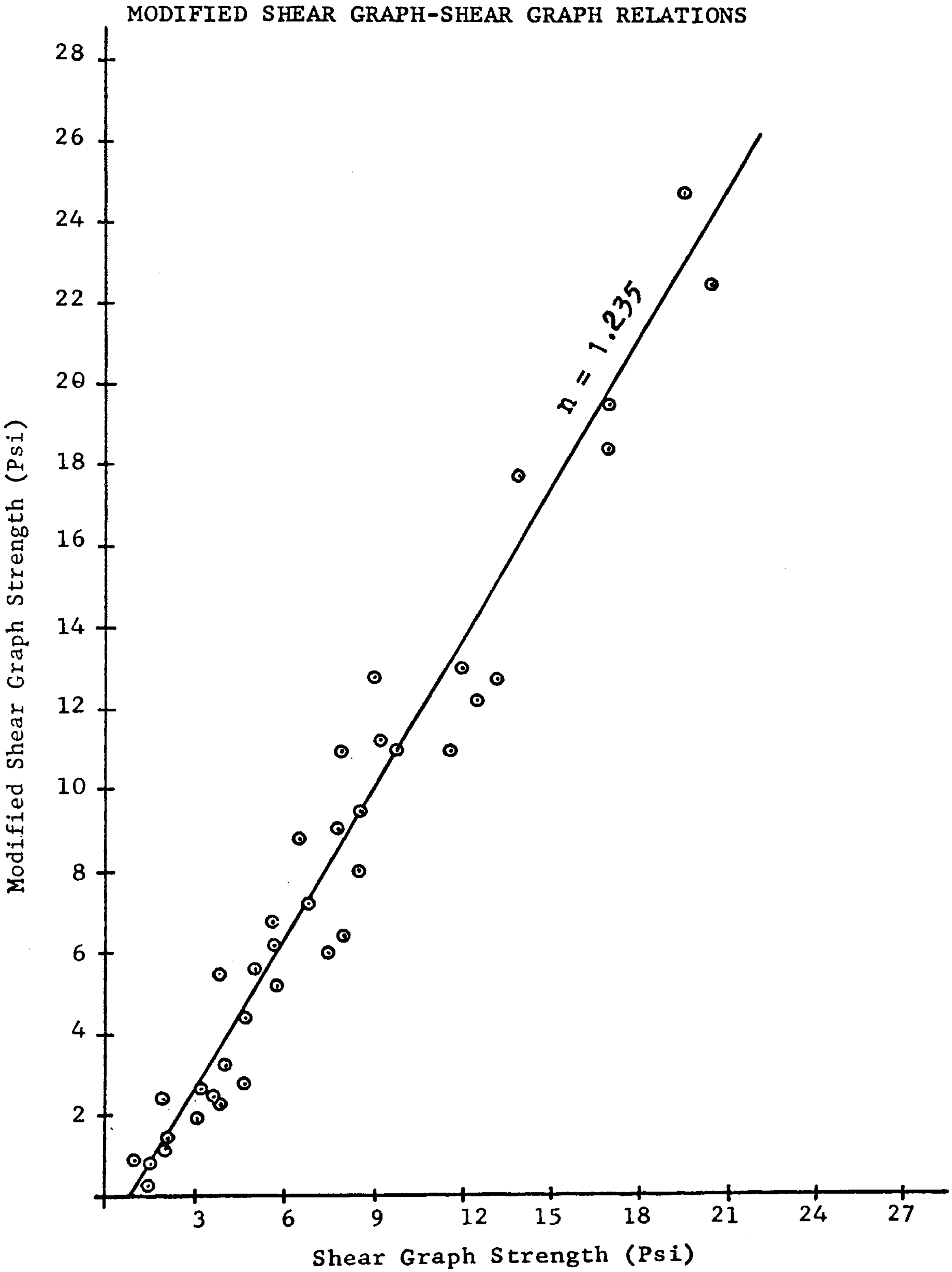


FIGURE 24. REMOLDED SOIL

MODIFIED SHEAR GRAPH-UNCONFINED RELATIONS

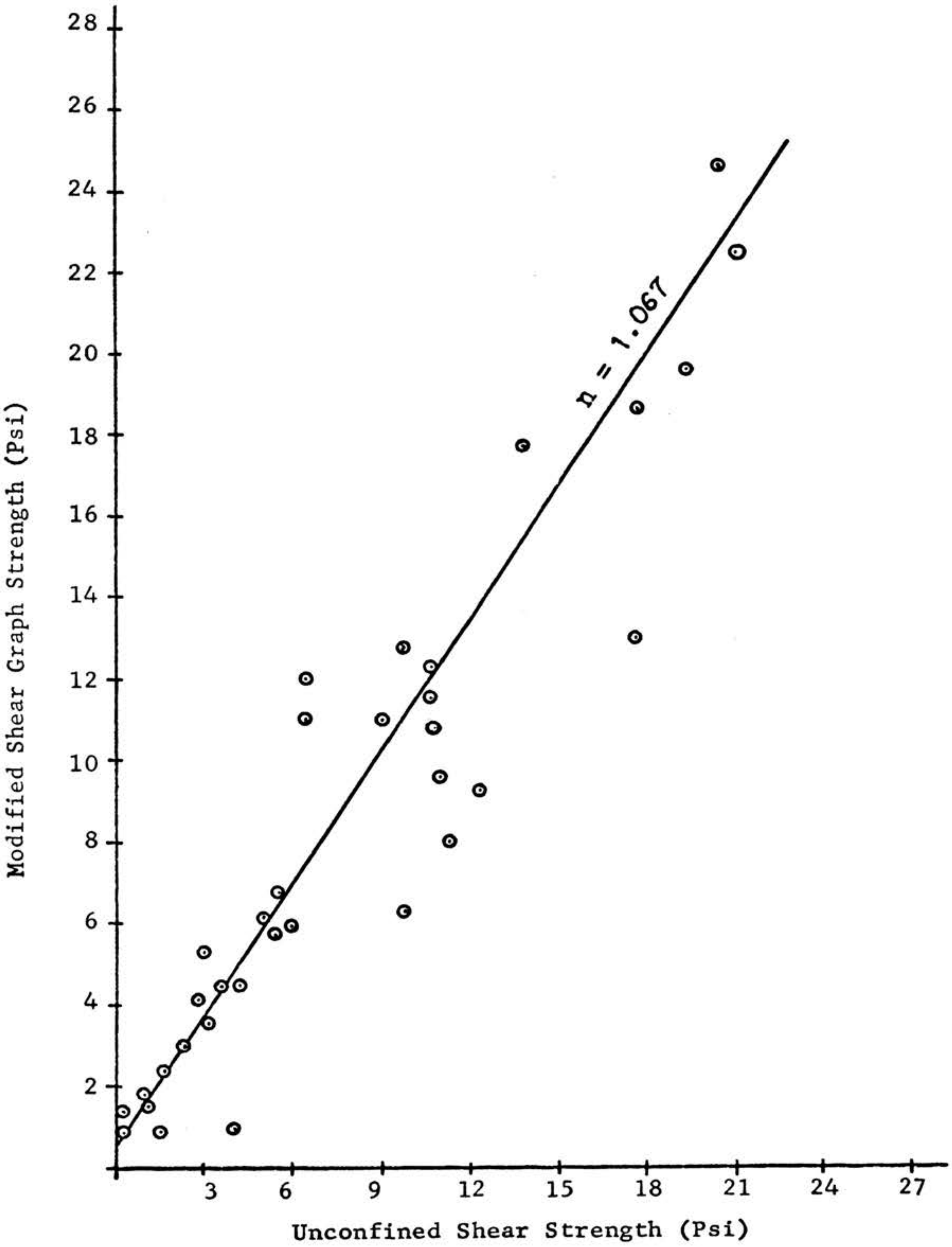


FIGURE 25. REMOLDED SOIL

From this relation it appears that the modified shear graph yields values 1.067 times those as indicated by the unconfined compression test.

Analysis of the test results indicate that the modified shear graph yields values which compare very favorably with the laboratory vane shear. In both the undisturbed and remolded tests, the values of shear strength as indicated by the modified shear graph were more closely related to the laboratory vane shear than with either of the other two pieces of equipment. Since both of these types of test equipment are similar in design and principle of operation, it can be assumed that the results would be comparable. The results of this investigation have proven that the modified shear graph is a reliable piece of test equipment and can be expected to yield consistent results.

Before additional investigations are conducted however, the detachable recording system should be constructed of metal and permanently fixed to the shear graph itself. This would tend to reduce the error involved in using temporary detachable parts. All those parts which were constructed of plexiglass should be constructed of steel or aluminum as the case warrants. This would include the protractor, indicator arms and base plate and spring head. Before tests are performed on granular materials the brass vane should be replaced with one constructed from stainless steel.

The rate of application of torque is very important, consequently, a timing device should be included with the model. This could be a stopwatch or smaller version of a kodak timer. A watch with a large second hand could be used if nothing else is available however.

VI. CONCLUSIONS

The objective of this research was to investigate methods of improving the standard soil shear graph in order that it would yield more reliable results when used to test cohesive soils. The conclusions made as a result of analyzing the test results are:

(1) The modified shear graph as described herein produces results which compare very favorably with the results obtained when using the laboratory vane shear.

(2) The modified shear graph yields higher values of shear strength of undisturbed soil than the unconfined compression test.

(3) The calibrated spring which is part of the standard soil shear graph is too strong for use in testing cohesive soils of high moisture content.

(4) The calibrated spring is too weak for obtaining values of shear strength greater than 25 psi.

(5) The modified shear graph requires no changing of recording paper as in the case of the standard model.

(6) The values of shear strength as indicated by the modified shear graph varied very little, if any, with variable normal loads. This is probably due to the large contact area of the base plate and small end area of the vanes.

VII. RECOMMENDATIONS

As a result of this investigation the following design changes are recommended:

1. Construct the indicator arms, spring head and base plate using stainless steel to increase their durability and wearability.

2. Replace the plexiglass protractor with a permanently mounted base and adjustable scale for zeroing purposes.

3. Mount a permanent normal load scale to the drum or machine a scale on the drum itself.

4. Replace the brass vane with one of stainless steel or spring steel. This should be machined from a solid piece of stock to include the desired length of shaft.

5. Groove the handle and knurl the shaft to provide a better grip when testing soils of high shear strength.

6. Provide additional calibrated springs to increase the testing range capability of the shear graph.

To complete the evaluation of the modified shear graph additional research should be conducted to include the following:

1. A greater number of tests should be run on a wide range of soils to include both cohesionless, and cohesive-friction soils.

2. Compare the modified shear graph with the triaxial test for soils with an appreciable angle of internal friction.

3. Determine the most appropriate rate of strain to be used when conducting tests.

4. Vary the size of the vanes and length of shaft so that several tests at different depths can be conducted with in the same location.

5. Conduct tests using different $\frac{H}{D}$ ratios to determine the most favorable ratio.

APPENDIX A

THEORY OF SOIL FAILURE WITH THE SHEAR GRAPH

THEORY OF SOIL FAILURE WITH THE SHEAR GRAPH

It is commonly assumed that soil shearing stress after failure is not strain dependent and that the soil is in the plastic state so that the shear stress remains constant with varying strain. This fact was used in calibrating the shear graph using the following procedure.

After failure has been reached, the soil on the failure plane has gone into the plastic state. When this occurs the assumption can be made that a constant ultimate shear stress (τ_u) exists across any radius of the shear head (Figure 26a). The resisting moment of the shear stress is found by integrating over the shear area where r is the distance from the center to the differential element and r_o is the radius of the shear head (the subscript $_u$ indicates ultimate values).

$$M_u = \text{Stress} \cdot \text{Area} \cdot \text{Moment Arm}$$

$$dM_u = \tau_u \cdot 2\pi r dr \cdot r$$

$$= 2\pi\tau_u r^2 dr$$

$$M_u = 2\pi\tau_u \int_o^{r_o} r^2 dr$$

$$= 2\pi\tau_u \frac{r_o^3}{3}$$

$$M_u = 2/3\ \pi\tau_u r_o^3$$

The effective radius of the area (r_e) can be found by dividing the resisting moment by the total shear force (F), $r_e = 2/3r_o$. Knowing the shear force and the effective radius of the force makes it possible to calibrate the shear graph record in terms of ultimate shearing stress.

The problem of determining peak shear stress (τ_p) is more complicated, particularly since the theory of linear stress-strain relationships prior to initial failure has not been proven. If this theory is assumed to be correct and if it is further assumed that the arrangement of the shear head vanes is adequate to produce uniformly increasing strain across any radius, and that normal stress is uniform, a linear stress distribution may also be assumed (Figure 26b). It also follows that the outermost grain in the torsional shear test area will be the first to reach the maximum peak shear stress. The resisting moment of a stress distribution such as this will not be the same as in the plastic case. Intermediate values of stress depend upon the distance from the center of rotation and reach a maximum of τ_p at r_o. The resisting peak moment for the triangular distribution is found by integrating.

$$M_p = \text{Stress}.\text{Area}.\text{Moment Arm}$$

$$dM_p = \tau .2\pi\, rdr.r$$

$$\tau = r/r_o \tau_p$$

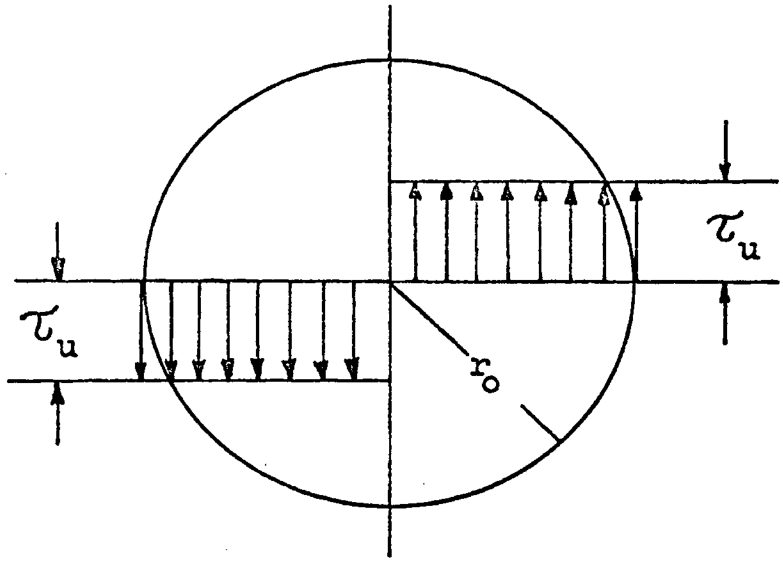
$$dM_p = r/r_o \tau_p .2\pi\, rdr.r$$

$$M_p = 2\pi \tau_p/r_o \int_o^{r_o} r^3 dr$$

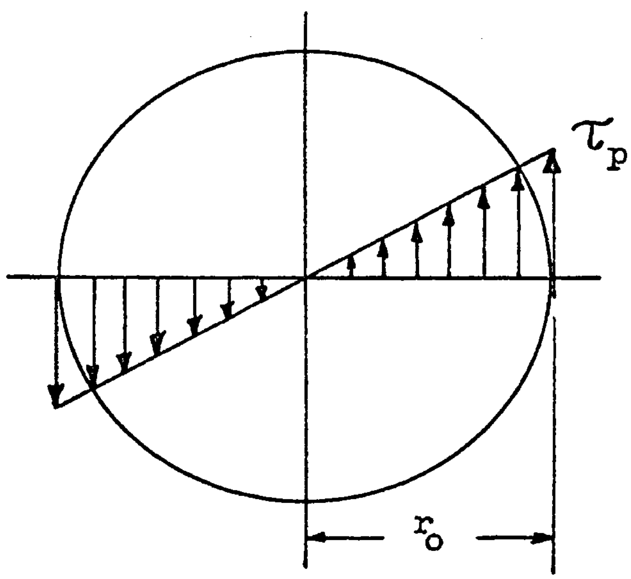
$$M_p = 1/2\, \pi \tau_p r_o^3$$

Although this equation describes the resisting moment when the outer fiber has reached its peak shear stress, the shear graph is not calibrated to record the true value of τ_p with the assumed triangular stress distribution.

For some soils when failure is reached, most of the shear area has gone plastic making the ultimate shear stress calibration valid. However, most soils at the instant of failure will exhibit a stress distribution that is a combination of both ultimate and peak values. As the outer fibers are strained a peak stress is reached, but this



a. Complete Plastic Distribution



b. Non-Plastic Stress Distribution

STRESS DISTRIBUTION

FIGURE 26.

stress is not enough to cause failure over the entire area. Therefore, the outer fibers pass into a plastic state and the peak stress progresses toward the center of rotation until failure occurs with the inner area under the influence of peak stress and the outer area having ultimate shear stress.

APPENDIX B

THEORY OF SOIL FAILURE WITH THE MODIFIED SHEAR GRAPH

THEORY OF SOIL FAILURE WITH THE MODIFIED SHEAR GRAPH

The theory of soil failure with the shear graph as presented in Appendix A is not changed by replacing the shear head with the base plate and vane apparatus. The geometric design and arrangement of the vane changed, however, the principle remains the same.

Shearing resistance is assumed to develop on a cylindrical surface circumscribed by the vertical edges as the vane is rotated. In addition to the vertical walls of the cylinder, a conical surface is described by the bottom edges of the vane. It is further assumed that the shearing resistance has constant magnitude over the cylindrical surface. Two assumptions are made concerning the shearing resistance over the conical surface: (a) the resistance is constant and equal in magnitude to that on the cylindrical surface or, (b) the resistance increases linearly from 0 at the axis to a maximum at the radius of the cylindrical surface, (is proportional to the radial distance).

The following analysis of the resisting moment developed on the conical surfaces, and the total resisting moment developed by the soil is extracted from Gray's Analysis of Loads on Vanes.[6]

The resisting moment (M_c) of the shear stress (τ max) developed on the cylindrical surface is given by:

$$M_{cyl} = \text{Area} \cdot \text{Moment Arm} \cdot \text{Stress}$$

$$M_{cyl} = 2\pi RH \cdot R \cdot \tau\ \max$$

where: H = Height of vane.

R = Radius of vane.

The resisting moment developed on the conical surface (M_{c_1}) when the shear stress is assumed to be constant over the conical surface is given by:

$$M_{c_1} = \text{Area} \cdot \text{Moment arm} \cdot \text{Stress}$$

$$M_{c_1} = \frac{1}{2}(2\pi R)\sqrt{R^2 + t^2} \cdot \text{Moment arm} \cdot \tau_{max}$$

where: t = Height of cone

$\sqrt{R^2 + t^2}$ = Slant height of cone

If the moment arm of the shear force is assumed to act through the center of gravity of the triangular surface, or one third the height (t) from the base, then:

$$M_{c_1} = \pi R \sqrt{R^2 + t^2} \cdot \frac{2}{3} R \cdot \tau_{max}$$

where: $\frac{2}{3} R$ = Moment arm of shear force

The total resisting moment (M_T) developed by the soil assuming constant shear force on all surfaces becomes:

$$M_T = M_{cyl} + M_{c_1}$$

$$M_T = 2\pi R^2 H \tau_{max} + \frac{2}{3}\pi R^2 \sqrt{R^2 + t^2}\, \tau_{max} \frac{(H)}{(H)}$$

$$M_T = M_{cyl}\left(1 + \frac{\sqrt{R^2 + t^2}}{3H}\right)$$

If the shear stress is assumed to vary from zero at the apex to a maximum at the radius of the cylinder then the resisting moment (M_{c_2}) developed on the surface of the cone becomes:

$$M_{c_2} = \text{Area} \cdot \text{Moment arm} \cdot \text{Stress}$$

$$M_{c_2} = \frac{1}{2}(2\pi R)\sqrt{R^2 + t^2} \cdot \frac{R}{2} \cdot \tau_{max}$$

where: $\frac{R}{2}$ = Moment arm of the shear force

The total resisting moment (M_T) developed by the soil assuming the shear stress is proportional to the radial distance is given by:

$$M_T = M_{cyl} + M_{c_2}$$

$$M_T = 2\pi R^2 H \tau_{max} + \frac{1}{2}\pi R^2 \sqrt{R^2 + t^2}\, \tau_{max} \frac{(2H)}{(2H)}$$

$$M_T = M_{cyl} \left(1 + \frac{\sqrt{R^2 + t^2}}{4H} \right)$$

By using the expressions derived above and substituting the dimensions of the test vane, the equations for shear stress become:

$$M_{cyl} = 2\pi R^2 \quad H \tau_{max} = 1.366\, \tau_{max}$$

where: $R = 0.468$ inches

$t = 0.281$ inches

$H = 0.968$ inches

Assuming the shear stress is constant on all surfaces the total resisting moment becomes:

$$M_T = M_{cyl} \left(1 + \frac{\sqrt{R^2 + t^2}}{3H} \right) = 1.366\ (1 + 0.183)\, \tau_{max}$$

$$M_T = 1.581\, \tau_{max} \text{ and,}$$

$$\tau_{max} = 0.632\, M_T$$

Assuming the shear stress is proportional to the radial distance the total resisting moment becomes:

$$M_T = 1.366\ (1 + 0.137)$$

$$M_T = 1.518\, \tau_{max} \text{ and,}$$

$$\tau_{max} = 0.659\, M_T$$

Gray has shown that the difference in the values of shear strength measured using either assumption to be less than 1 percent of the total resistance. The results of this investigation indicate that the percent difference between these two equations varies from less than 1 percent to a maximum of 7.4 percent.

There is an error in this analysis because the area of the conical surface is not a lineal function of the radial distance or height of the cone. To obtain the actual true value of shearing resistance developed requires integrating the area and shear stress, with respect to the radius and height of the cone. However, as the physical characteristics of soil vary within short distances, the difference between the actual shear stress and that determined by the above equations would be insignificant and the error negligible.

One additional consideration must be discussed before the theory of the modified shear graph can be fully understood. As the ratio of the cylinder height to cone height becomes smaller, the error in using Gray's analysis becomes greater. In other words, as the ratio of the conical to the cylindrical area becomes larger the effect of the size of the conical area on the resisting moment becomes greater. Consequently the error increases accordingly. This error approaches zero when the ratio of vane height to vane width equals two. For ratios of $\frac{H}{D} < 1$ the error becomes of sufficient magnitude to warrant a complete analysis of the actual and theoretical values, and a determination made as to which value to use.

APPENDIX C

TABULATION OF TEST DATA

TABLE IV. UNDISTURBED CLAY TEST RESULTS

Test No.	Moist Cont (%)	Laboratory Vane Shear Strength psi	Shear Graph Strength psi	One-half Unconfined Strength psi	Modified Shear Graph Strength psi
1	74.5	3.6	4.8	2.1	6.1
2	66.4	3.8	4.0	4.0	5.7
3	65.0	3.5	3.8	3.2	6.3
4	66.5	3.5	4.8	4.1	5.6
5	61.7	4.4	5.3	3.5	5.4
6	81.4	3.9	5.2	1.5	5.3
7	82.4	3.9	3.6	1.6	5.9
8	74.5	4.16	2.5	2.0	6.3
9	72.5	3.5	2.9	2.3	8.0
10	70.5	3.1	3.4	1.4	7.1
11	73.5	3.3	3.2	2.1	6.2
12	84.4	3.9	3.4	1.4	5.7
13	64.6	3.5	2.2	3.2	5.7
14	71.7	4.5	6.1	1.6	5.6
15	76.8	3.6	4.7	2.4	6.4
16	59.2	4.6	2.4	3.5	5.5
17	65.3	4.4	5.6	4.1	5.5
18	65.3	4.4	5.6	4.1	5.5
19	69.0	3.7	4.4	2.1	5.2
20	65.3	4.0	5.0	5.1	7.7
21	72.7	3.7	2.7	2.3	7.9
22	58.5	4.1	4.7	2.6	4.6
23	64.1	3.5	2.2	4.3	5.0
24	82.2	3.9	3.6	1.5	5.6
25	66.5	3.5	4.8	3.2	5.3

TABLE V. REMOLDED CLAY TEST RESULTS

Test No.	Laboratory Vane Strength Psi	Shear Graph Strength Psi	Modified Shear Graph Strength Psi	One-half Unconfined Compressive Psi	Moisture Content Psi	Dry Density Pcf
1	1.22	1.33	---	----	65.5	60.9
2	2.15	2.0	2.43	1.72	54.6	68.2
3	0.8	1.3	---	----	67.0	59.2
4	0.78	0.75	---	----	64.3	60.1
5	0.73	1.1	0.95	1.64	64.2	61.1
6	1.39	1.5	1.82	1.06	48.6	72.2
7	3.68	4.7	4.49	3.54	44.6	77.5
8	4.72	7.5	6.03	6.0	37.6	82.5
9	0.91	1.3	1.00	---	65.4	60.9
10	1.04	1.4	1.39	0.262	65.5	60.6
11	0.47	1.2	.88	0.344	61.8	61.0
12	1.21	2.0	1.74	1.2	56.4	64.9
13	3.0	4.7	4.2	2.79	45.7	75.5
14	0.75	1.5	.96	0.71	63.6	58.9
15	1.69	3.6	2.5	0.418	59.4	65.8
16	0.99	1.7	1.33	0.26	65.2	62.5
17	0.95	1.5	1.42	0.278	70.0	60.6
18	9.6	8.1	12.15	6.4	37.3	82.3
19	0.76	1.0	.86	0.516	58.6	62.3
20	1.05	1.4	1.1	3.98	63.4	61.7
21	1.46	2.0	1.8	1.52	52.1	67.6
22	2.46	3.8	3.0	2.37	47.5	73.7
23	3.68	4.7	4.49	4.2	44.0	80.5

Test No.	Laboratory Vane Strength Psi	Shear Graph Strength Psi	Modified Shear Graph Strength Psi	One-half Unconfined Compressive Psi	Moisture Content Psi	Dry Density Pcf
24	6.85	8.5	9.3	1.7	14.8	.
25	1.22	9.3	1.7	1.63	62.8	62.4
26	2.08	2.1	2.35	1.75	54.6	67.7
27	2.5	3.2	3.1	2.63	51.4	69.7
28	3.12	4.0	3.55	3.16	50.0	72.8
29	8.27	11.7	9.55	11.0	31.8	89.0
30	11.1	12.5	12.25	10.7	31.3	89.8
31	9.7	7.9	10.95	6.4	37.6	82.2
32	10.6	12.0	13.2	13.2	29.4	90.8
33	9.75	9.0	10.8	10.8	34.0	86.2
34	6.6	6.8	6.75	5.45	36.0	83.5
35	8.9	9.7	10.95	8.85	32.1	88.2
36	2.6	3.0	2.77	1.91	44.4	74.4
37	2.57	2.8	2.92	2.37	48.6	70.6
38	4.95	5.6	6.72	2.76	38.7	79.4
39	11.8	13.3	12.85	9.8	32.6	88.0
40	16.7	14.0	17.8	13.8	31.1	90.0
41	19.8	20.6	22.6	21.9	28.3	93.8
42	0.76	1.2	0.94	0.41	61.9	60.8
43	0.90	1.3	1.4	0.6	54.7	65.2
44	1.39	1.8	1.96	1.36	48.6	70.0
45	1.73	1.9	1.97	1.09	58.0	64.8
46	6.25	5.7	6.2	5.11	35.2	84.1
47	4.38	5.8	5.29	3.14	39.4	79.3

Test No.	Laboratory Vane Strength Psi	Shear Graph Strength Psi	Modified Shear Graph Strength Psi	One-half Unconfined Compressive Psi	Moisture Content Psi	Dry Density Pcf
48	4.51	5.0	5.69	5.4	35.0	83.3
49	6.25	6.5	8.78	3.28	35.7	84.0
50	6.42	8.5	8.07	11.3	33.4	86.1
51	7.02	7.8	9.3	12.4	34.7	85.6
52	10.9	10.2	11.45	10.7	32.4	89.0
53	17.3	17.0	18.55	17.7	29.8	92.5
54	21.2	17.1	19.7	19.3	28.2	94.3
55	20.4	19.6	24.8	20.3	28.1	94.7
56	2.43	2.4	2.64	2.14	46.1	72.9
57	2.7	3.0	3.22	1.93	34.3	73.6
58	3.06	3.9	3.54	3.12	50.0	72.7
59	0.69	1.2	0.87	0.36	65.6	59.9
60	0.69	1.3	0.82	0.75	64.3	59.4
61	1.32	1.5	1.25	0.97	61.9	63.0
62	1.56	2.0	1.80	1.28	57.9	66.5
63	2.08	2.3	2.35	1.53	50.3	69.3
64	2.5	3.5	2.97	2.06	48.6	70.6
65	4.9	5.0	6.28	9.84	38.7	80.0

BIBLIOGRAPHY

1. Nickerson, C. R. (1965). The Shear Graph and Cohesive Soil. Thesis, University of Missouri at Rolla, 57 p. (with 21 figr., 5 tables.)

2. Cadling, L. and Odenstad, S. (1950). The Vane Borer. Proc. Royal Swedish Geotechnical Institute (Stockholm), Vol. 2.

3. Carlson, L. (1948). Determination In Situ of the Shear Strength of Undisturbed Clay By Means of a Rotating Auger. Proc. Second International Conference on Soil Mechanics and Foundation Engineering. Vol. 1, p. 266.

4. Symposium on Vane Shear Testing of Soils (1957). ASTM Spec. Tech. Pub. No. 193. p. 61, Philadelphia.

5. Evans, I. and Sherratt, M. A. (1948). A Simple and Convenient Instrument for Measuring the Shearing Resistance of Clay Soils. Journal of Scientific Instruments. Vol. 25, pg. 412-414.

6. Gray, H. (1957). Field Vane Shear Tests of Sensitive Cohesive Soils. Trans. American Society of Civil Engineers. Vol. 122, p. 844.

7. Vey, E. (1957). Vey on Vane Test (Discussion). Trans. American Society of Civil Engineers. Vol. 122, p. 858.

8. U. S. Department of Interior, Bureau of Reclamation. Earth Manual, 1962. p. 563.

9. Liu, T. K. and Thornburn, T. H. (1963). Investigations of Surficial Soils. ASTM Spec. Tech. Pub. No. 351, p. 44.

10. U. S. Army Engineer Waterways Experiment Station. Technical Report No. 3-639, 1964. Strength-Moisture-Density-Relations of Fine-Grained Soils in Vehicle Mobility Research.

11. Payne, P. C. J. and Fountaine, E. R. (1952). Measuring the Shear Strength of Soils. Journal of Soil Science, Vol. 3, No. 1.

12. Cohron, G. T. (1962). The Soil Shear Graph. American Society of Agricultural Engineers. Paper No. 62-133.

13. Terzaghi, K. and Peck, R. B. (1948). "Soil Mechanics in Engineering Practice". John Wiley and Sons, Inc., New York, p. 31.

14. Olson, R. E. (1964). Class Notes. Notes on Shear Strength.

15. Sowell, L. C., Jr. (1964). Compacting Characteristics of Fine Grained Soils. Thesis, School of Mines and Metallurgy of University of Missouri.

16. Jumikis, A. E. (1962). "Soil Mechanics". D. Van Nostrand Co., Inc., Princeton, New Jersey.

17. Scott, R. F. (1963). "Principles of Soil Mechanics". Addison-Wesley Pub. Co., Inc., Reading, Mass.

18. Peck, R. B., Hanson, W. E. and Thornburn, T. H. (1953). "Foundation Engineering". John Wiley and Sons, Inc., New York.

19. Hvorslev, M. J. (1939). Torsion Shear Tests and their Place in Determination of the Shearing Resistance of Soils. Symposium on Shear Testing of Soils, ASTM Spec. Tech. Pub. No. 39, Philadelphia.

20. Bennet, G. B. and Mecham, J. G. (1953). Use of the Vane Borer on Foundation Investigations of Fill. Proc. Highway Research Board, Vol. 32.

21. Housel, W. S. (1959). Dynamic and Static Resistance of Cohesive Soil 1846-1958. ASTM Spec. Tech. Pub. No. 254, Papers on Soils, 1959 meeting.

22. Winterkorn, H. F. and Tscheboratioff, G. P. (1947). Sensitivity of Clay to Remolding and Its Probable Causes. Proc. Highway Research Board, Washington, D. C., Vol. 27, p. 435.

23. Symposium on In-Place Shear Testing of Soil by the Vane Methods (1957). ASTM Spec. Tech. Pub. No. 1943, Philadelphia.

24. Casagrande, A. and Shannon, W. L. (1948). Stress-Deformation and Strength Characteristics of Soils Under Dynamic Loads. Proc. Second International Conference on Soil Mechanics and Foundation Engineering, Vol. V, p. 29-34.

25. Teng, W. C. (1962). "Foundation Design". Prentice-Hall, Inc., Englewood Cliffs, N. J., p. 13-18.

26. Tschebotarioff, G. P. (1951). "Soil Mechanics Foundations, and Earth Structures". McGraw-Hill Book Co., Inc., New York.

27. Taylor, D. W. (1948). "Fundamentals of Soil Mechanics". John Wiley and Sons, Inc., New York.

VITA

Vernelle Truman Smith was born on August 16, 1936 in Norwich, New York. He received his primary and secondary education at Westmoreland, New York, graduating from Westmoreland Central High School in June, 1954. His college education was received at Utica College of Syracuse University and Michigan Technological University. He received the degree of Bachelor of Science in Civil Engineering from Michigan Technological University in 1965.

He was commissioned into the United States Army Corps of Engineers as a Second Lieutenant in 1959 and has been on continuous active duty since that time. Presently, he is a Captain in the Regular Army. Duty Assignments include tours at Fort Belvoir, Virginia, Fort Benning, Georgia, Fort Jackson, South Carolina and Korea.

In September 1964 he was assigned to the University of Missouri at Rolla as a candidate for the degree of Master of Science in Civil Engineering.

He is married to the former Nancy C. Malone of Belton, South Carolina. They have one daughter, Deborah Annette.